# Performance of Ductile Reinforced Concrete Moment Resisting Frames Subject To Earthquake Actions

*A thesis*
*Submitted in partial fulfilment*
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*of*

**Master of Engineering (Civil)**
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by
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Supervised by
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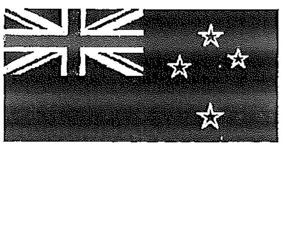
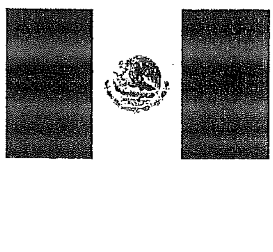
This project was supervised by Professors Athol Carr and Richard Fenwick. Their experience and knowledge never ceased to amaze me. Thank you for treating me like a friend. Your contribution to the development of science is invaluable.

I would like to acknowledge the constant support of the people from "*Alonso y asociados*", specially Javier who has been a great guide to my professional life. I admire the ethics and passion that you put in your work. I look forward to continue learning from you.

I would also like to thank this beautiful country and all the people that I met while at it. The contact with such a diversity of cultures and thoughts changed my life in the best possible way. I will always remember New Zealand and its people with gratitude.

Special recognition is given to the trees that fell so that I could print this report. I promise to repair the damage caused. I would also like to thank the big blue sea for being the place where I can drown all my problems and make my spirit rise.

And at last but not least, I would like to thank my mother, my father, my brother and my sister who supported me all the way and never stopped reminding me that I have people in the world that care for me. I love you.

# INDEX

**ERRATA**

In checking the thesis it was found that there was an error in the calculation of the column strengths designed by the NZS 1170.5 approach. The method of establishing the required strengths is correctly described in the text but the column strengths used in the analyses are typically 20 to 30 percent lower than the intended values.

## ABSTRACT

It has been shown that the strengths specified in the Loadings Standard NZS 4203:1992 (Standards New Zealand 1992) to resist seismic actions are low when compared with major international design codes (Fenwick and Davidson 1994; Fenwick et al. 2002). Few modifications to these low strengths other than an increase in the minimum permissible base shear have been made in the draft revision of the Standard, NZS 1170.5. Furthermore, the design procedure to allow for higher mode effects in multi-storey structures subject to dynamic forces was developed in the 70's using a limited number of non-linear time-history analyses with a bilinear hysteretic rule and in most cases neglecting P-delta effects.

In this work, a four storey, a six storey, and two twelve storey buildings, in which the resistance to lateral forces is provided by concrete moment resisting frame structures were designed and analysed.

Through a series of non-linear time-history analyses using a Takeda hysteretic rule and considering P-delta effects, three main objectives were studied. The first objective was to investigate if the strengths given to beams and columns met the objectives set by the Loadings Standard (Standards New Zealand 1992). The second objective was to examine how well the method of determining column actions from the NZS 3101:1995 (Standards New Zealand 1995) works when using the lateral loading specified in NZS 4203:1992 (Standards New Zealand 1992) and the draft provisions of the proposed Loadings Standard NZS 1170.5 and the third objective was to compare the performance of multi-storey moment resisting frame buildings were columns are modelled as:

- Elastic responding columns except at the base,
- Columns designed to meet the minimum requirements as given in NZS 3101:1995 (Standards New Zealand 1995), and
- Columns designed to meet the minimum strength requirements as defined in the 2004 draft of NZS 1170.5 where limited protection to plastic hinge formation is given.

The influence of the choice of hysteretic rule was assessed and in general, the structures studied performed in a satisfactory manner due to the use of a more realistic hysteretic model. The individual results from the non-linear time-history analyses were very scattered making the structures reach critical performance levels with some of the selected ground acceleration records and poor performance was observed for structures analysed using a 2,500 year return period earthquake.

It was also shown that P-delta effects have a significant influence to the response even for the four and six storey structures and concluded that P-delta effects should always be included in the design and analysis of structures.

# 1. OBJECTIVES

With the introduction of the Loadings Standard NZS 4203:1992 (Standards New Zealand 1992), there was a marked reduction in the minimum strength required in multi-storey buildings. It has been shown that the strengths specified in this code to resist seismic actions are low when compared with major international design codes (Fenwick and Davidson 1994; Fenwick et al. 2002). Few modifications to these low strengths other than an increase in the minimum permissible base shear have been made in the draft revision of the Standard, NZS 1170.5. The first objective of this work is to investigate if these strengths are adequate to meet the stated objectives of the Standard.

To design a ductile multi-storey moment resisting frame structure, it is essential to provide its columns with sufficient strength to prevent the premature formation of a column sway mechanism. In the 1970s a method was developed and adopted in the structural concrete Standard NZS 3101:1985 and 3101:1995 (Standards New Zealand 1995). This approach introduced the concept of a dynamic amplification factor, which was applied to the column moments to allow for "higher mode effects", which cause the distribution of column actions to diverge from those found in a static elastic analysis. However, due to technology restrictions at that time, the dynamic amplification factor was estimated through a limited number of non-linear time-history studies using a bi-linear hysteretic model and without considering P-delta effects (Park 1995). The second objective is to examine how well this method of determining column actions works when using the lateral loading specified in NZS 4203:1992 (Standards New Zealand 1992) and the draft provisions of the proposed Loadings Standard NZS 1170.5 with a more realistic hysteretic model and the inclusion of P-delta actions in the analysis.

When the proposed new Loading Standard NZS 1170.5 was being developed, an anomaly was apparent in the current requirements for columns. In ductile structural steel moment resisting buildings much lower strengths were required than the corresponding levels in reinforced concrete frame buildings. In an attempt to remove this anomaly, a second method of defining the design actions in columns was introduced. The intention was that this could be used for both structural steel and reinforced concrete multi-storey moment resisting frame buildings. With this approach, in a major earthquake plastic hinges could be expected to form at several levels over the height of the columns, while with the approach given in NZS 3101:1995 (Standards New Zealand 1995) plastic hinges would only be expected at the base of the columns and in the top storeys. In the revised edition for the NZS 3101:2005 (Standards New Zealand 2005), it is intended to give the designer the option to select either method to determine the required column strength. However, when plastic hinges are allowed, it should not be overlooked that potential plastic hinge regions need to be detailed accordingly so that proper confinement, anti-buckling reinforcement and adequate shear strength is provided by the transverse reinforcement and lap splices are located away from the potential plastic hinge zones.

The third objective is to compare the performance of multi-storey moment resisting frame buildings where the columns are modelled as:

1. Elastic responding columns except at the base,
2. Columns designed to meet the minimum requirements as given in NZS 3101:1995 (Standards New Zealand 1995), and
3. Columns designed to meet the minimum strength requirements as defined in the 2004 draft of NZS 1170.5 where limited protection to plastic hinge formation is given.

## 2. LITERATURE REVIEW

Research was undertaken by Professors Paulay and Carr, together with their post graduate students at Canterbury University in the mid-seventies to define a design procedure for reinforced concrete moment resisting frames subject to earthquake actions which complied with the capacity design principles. To do this the over-strength of materials and the dynamic effects of structures subject to earthquakes had to be fully understood. One of the first efforts to quantify the effects of dynamic amplification to the response of structures was Row (Row 1973). This was continued by Kelly (Kelly 1974). In his work, Kelly studied a 6 and a 12 storey buildings using two ground acceleration records with duration of 6 seconds or less for each structure, using bilinear hysteretic models and neglecting P-delta effects. The recommendation from his work to determine column design moments in accordance with the dynamic effects was that *"for buildings of six storeys or less, each column should be designed for 65% of the sum of beam over-strength capacities and for buildings with more than six storeys each column section should be designed for 85% of the sum of the beam over-strength capacities" (Kelly 1974)*. Jury (Jury 1978) continuing the work from Kelly established the concept of the dynamic amplification factor that is still used in the current Concrete Structures Standard 3101:1995 (Standards New Zealand 1995). In this work, two six storey, a twelve and an eighteen storey buildings were analysed using 10 seconds of two ground acceleration records for each structure, using a bilinear hysteretic rule. In this study P-delta effects were recognized as significant but little distinction was given in the results to account for them. From this it was concluded that a dynamic amplification factor of 1.2 at the first level appeared to be low but a smooth transition from the base of the column where no dynamic effects are present to higher levels was needed. It was also noted that if the dynamic amplification factors were overestimated, it would lead to an increase in the demand at the base of the column. It was noted that the dynamic effects in the columns varied with height in the building ant they were smaller in the lower third of the structure. Further studies were made to include P-delta effects such as those performed by Tompkins (Tompkins 1980) who studied a twelve, a three and two six storey reinforced concrete frames. For this project the dynamic amplification factor was evaluated with the same equation as in the Concrete Structures Standard (Standards New Zealand 1995). These analyses were made using a bilinear hysteretic rule and with an inter-storey drift design limit of 1% and three ground acceleration records with duration of 10 seconds. The results for these analyses showed that the inter-storey drift limit of 1% was never exceeded. Suggestions from this study were made to increase the dynamic amplification value for low rise buildings. These studies were continued by Carr and Moss (Moss and Carr 1980), who performed non-linear time-history analyses on a 6, 12 and 18 storey structures considering P-delta effects, using a bilinear hysteretic model. It was found that if the maximum inter-storey drift without P-delta was less than 1.5%, the inclusion of P-delta for the 12 storey structure reduced the maximum inter-storey drift and if the inter-storey drift was higher than 2.0%, the maximum inter-storey drift would increase 1.50 times when compared to the maximum inter-storey drift from the analyses that did not include P-delta effects. As a final recommendation it was said that when designing according to the New Zealand design codes of the time, P-delta effects did not need to be considered.

Fenwick and Davidson with their post-graduate students at Auckland University (Chung 1993; Chung et al. 1991; Fenwick and Davidson 1991; Fenwick et al. 1992) continued with the research to asses the influence of P-delta effects in single degree of freedom and multi-storey structures. From these studies the simplified method to include P-delta effects in the design of structures described in the Loadings Standard (Standards New Zealand 1992) was defined and calibrated.

## 3. APPROACH TO PROJECT

The approach that has been adopted is to design and analyse a four storey building, a six storey building, and two twelve storey buildings, in which the resistance to lateral forces is provided by concrete moment resisting frame structures. These structures were designed following a capacity design approach (Paulay and Priestley 1992), so that accordingly to the strength of the beams, the columns were designed and modelled by the three different methods. The first method consists on modelling elastic columns except at the base. The second design method follows the NZS 3101:1995 (Standards New Zealand 1995) providing the columns with a high level of protection against plastic behaviour except at the base and the top floor. The third method is that described in the 2004, 1170.5 draft, which allows the formation of plastic hinging in the columns between the base and top floor. Once these models were completed, each was analytically tested with 8 different earthquake ground acceleration records through non-linear time-history analyses. Four of these ground acceleration records were modified so that their acceleration response spectra would match the design acceleration response spectra defined by NZS 4203:1992 (Standards New Zealand 1992) at all periods. The other four ground acceleration records were used as they were originally recorded but each separately scaled using the first scale factor k1, as outlined in the draft for NZS 1170.5 so that the difference between the acceleration response spectra resulting from these records and the design response spectra defined by the draft NZS 1170.5 is minimized over a period range in accordance to the draft standard.

From these time-history analyses, the resulting maximum inter-storey drifts, maximum storey displacements and maximum element curvatures were obtained and studied to assess the behaviour of the structures.

Using the four modified ground acceleration records, analyses have been performed on the six and one of the twelve storey structures to assess the influence that the choice of hysteretic model has on the analyses. This step was made so that the results given by these analyses could be compared to those performed in earlier research projects where simpler, less realistic hysteretic models were used.

A brief study is made of the six storey structure to outline the differences between structures designed to different ductility factors. Four concrete moment resisting frames were designed using structural ductility factors of 6, 4, 2 and 1 respectively. The response of these structures when subject to the four modified ground acceleration records was found. The relation between P-delta effects and the ductility factor was also studied.

# 4. INTRODUCTION

## *4.1 Post-elastic Mechanisms of Deformation of Earthquake Resisting Structures*

When designing structures, a high level of protection is required against undesired inelastic behaviour. In ductile frames, it is important to restrict hinging in columns away from the base (Paulay and Priestley 1992). There are two main reasons why such column hinging should be carefully considered. The first, which applies to steel and reinforced concrete buildings, is to prevent the premature formation of a column sway mechanism, as this can lead to a premature collapse of the structure. The second, related just to reinforced concrete buildings, is the need to consider the additional confinement that is required in columns in which plastic hinges form and the adverse influence that plastic hinge zones have on lap splices in columns. By performing a non-linear time-history analysis it can be seen that due to the dynamic effects the point of contra-flexure for a column in a given storey could shift and even disappear (Paulay 1979).

Figure 4.1 illustrates the possible post-elastic mechanisms for moment resisting frames subject to lateral load. The white circles represent the formation of a plastic hinge. If a column sway mechanism is formed, the rotations concentrate at the plastic hinges in a single storey. This could lead to premature collapse of the structure.

It should be noted that the mechanisms shown in Figure 4.1 are an idealisation coming from a static pushover analysis when the structure is subject to code type forces distributed to fit the first mode of vibration of the structure (Park 2003).

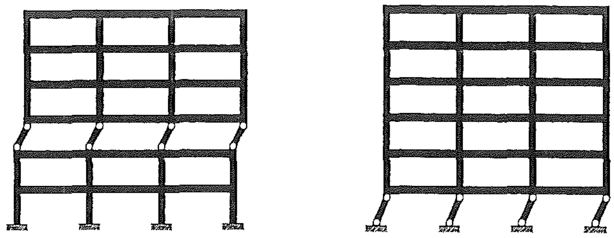
For a beam sway mechanism to form, the beams have to yield before the columns. To achieve this, the maximum likely strength of the beams (over-strength) is assessed and the columns are designed to sustain the maximum actions that can be transmitted to them. This capacity design process is known as a weak beam-strong column mechanism. It has generally been expressed by Equation 1.

$$\Sigma M_b < \Sigma M_c \qquad \text{(Eq. 1)}$$

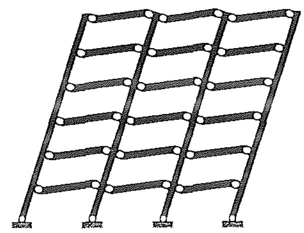
where $\Sigma M_b$ is the sum of the beam over-strengths that needs to be resisted by the columns at the beam-column junction being considered and $\Sigma M_c$ is the sum of the columns flexural capacity at this same junction. However, when P-delta actions and other effects are considered it can be shown that some additional column over-strength is required if a premature mixed beam-column sway mode (see figure 4.1) is to be avoided.

A beam sway mechanism is the most convenient when designing a moment resisting frame because moderate demands on the curvature ductility would be required on the plastic hinges forming at column bases and beams (Park and Paulay 1975). A mechanism of this type makes it easier to meet the demands at the plastic hinges if the structure is carefully detailed (Park 2001). Because of this the New Zealand Loadings Standard 4203:1992 (Standards New Zealand 1992) requires that columns in moment resisting frames to have sufficient strength to ensure a beam sway mechanism forms in preference of a column sway mechanism.

Column Sway mechanism



Beam Sway mechanism



Combined mechanism

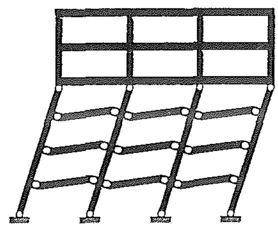


**Figure 4-1 Post-elastic mechanisms for moment resistant frames**

The New Zealand Loadings Standard NZS 4203:1992 (Standards New Zealand 1992) and Structural Concrete Standard NZS 3101:1995 (Standards New Zealand 1995) consider acceptable a column sway mechanism in two cases:

1. For one or two storey buildings a column sway mechanism is permitted;
2. For gravity-dominated frames of more than two levels, some columns may be permitted to form plastic hinges (combined mechanism) provided that the other columns in the storey remain elastic preventing the column sway mechanism from developing.

For structures with more than two storeys, hinging at the top storey columns is permitted.

A combined mechanism may form provided plastic rotations in hinge zones are small, which tends to be the case when the column hinges are separated by several storeys.

### *4.2 Capacity Design*

In order to limit the formation of plastic hinges so that the post-yield behaviour is consistent with the selected mechanism, and to avoid a shear or bond failure so that the displacement ductility capacity of the structure complies with the design requirements, a design method known as "capacity design" was developed.

The basis for the capacity design procedure were first published in 1969 in a paper for the New Zealand National Society for Earthquake Engineering (Hollings 1969) and further developed by Park and Paulay and Paulay and Priestley[1] (Park and Paulay 1975; Paulay and Priestley 1992). The steps to capacity design due to lateral forces are (Paulay and Priestley 1992) :

1. Select the potential plastic hinge zones.
2. Design and detail the zones to be capable to develop the required design strengths. Special consideration should be made when designing transverse reinforcement and as well the location of the lap splices in elements.
3. Assess the maximum likely strength (over-strength) that these zones will be capable of developing and hence transmit to the adjoining elements. Many factors are involved in the evaluation of the flexural over-strength of members. These include the variation of steel and concrete strengths, strain hardening or additional reinforcement that could be placed for construction purposes to satisfy minimum requirements or satisfy available bar size. The use of strength reduction factors and participation of non-structural members should be considered (Park 2003).
4. Design the rest of the structure so that its elements are capable of resisting the maximum actions induced when the elements with potential plastic hinge zones are taken to their maximum likely strength.

This approach ensures that the inelastic behaviour is limited to the plastic hinge zones and the formation of the failure mechanism selected by the designer.

[1] Information on the development of capacity design was taken from the article Park, R. (2001). "Improving the resistance of structures to earthquakes." *Bulletin of the New Zealand National Society for Earthquake Engineering*, 34(1), 1-40..

# 5. GENERAL DESIGN CONSIDERATIONS

## *5.1 General Description of the Structure*

Four different reinforced concrete moment resisting frames were designed, one each with 4 and 6 storeys and two with 12 storeys. The same simplified floor plan configuration, shown in figure 5.1, and site characteristics were used for all frames. These structures resist any lateral forces by two reinforced concrete frames on one direction and 2 structural concrete walls in the perpendicular direction. This configuration was idealized for simplicity in the analysis using a symmetric floor plan with negligible torsional effects and only low axial loads to act on the columns of the frames. The weight per floor is 4,400 kN which is carried mainly by the internal columns and the walls. The internal columns were assumed to be flexible. Thus the moment resisting frames carry their self-weight and provide all the lateral force resistance at right angles to the walls. The material properties for the moment resisting frames are listed in Table 5.1.

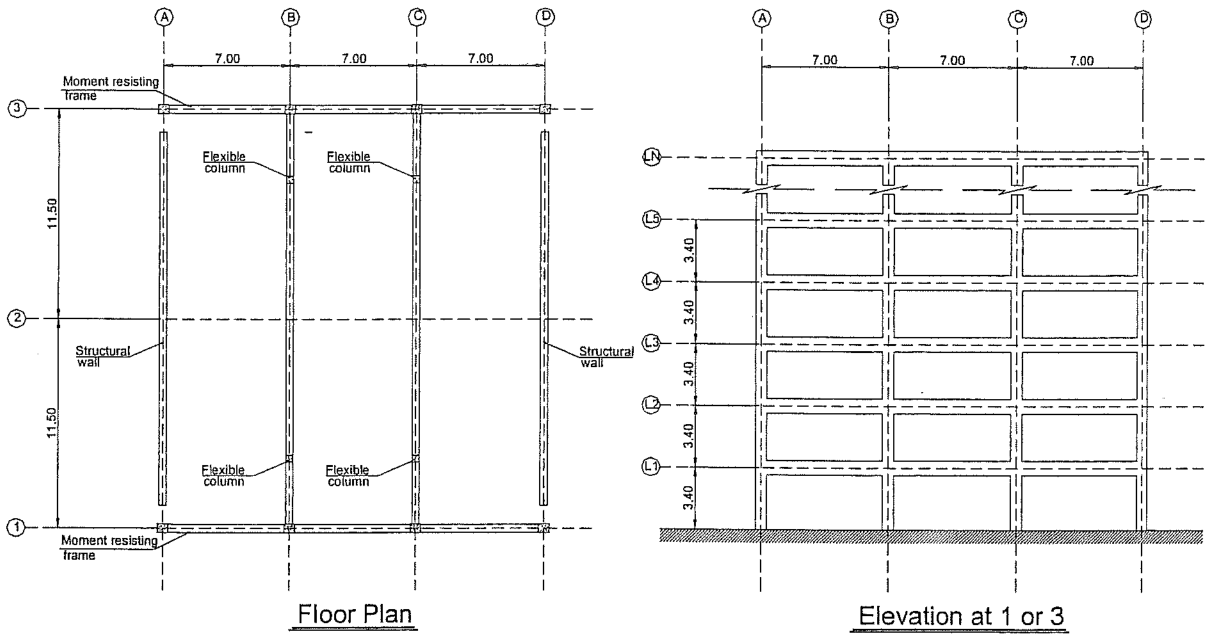



**Figure 5-1 General configuration of the structure (All units are in meters)**

**Table 5-1 Material properties**

| *Concrete* | *Longitudinal Steel* |
|---|---|
| $f'_c = 30$ MPa | $f_y = 300$ MPa |
| E = 25 GPa | |
| Weight per unit of volume: 23.5 kN/m$^3$ | |

*5.2 Site and Seismicity*

The assumed location is in Wellington area (Z = 1.2), on intermediate type soil as defined by the Loadings Standard (Standards New Zealand. 1992), with no near fault effects.

*5.3 Parameters of Equivalent Static Analyses*

For the equivalent static analysis, the design acceleration response spectra used was taken from the NZS 4203:1992 Loadings Standard (Standards New Zealand. 1992) corresponding to a ductility ($\mu$) of 6 on intermediate type soil scaled by a Zone factor of 1.2 and a Structural Performance Factor of 2/3, as required in the Standard.

*5.4 Parameters of Modal Response Spectral Analysis*

The acceleration response spectrum was used for both the equivalent static and modal response spectrum analyses. The Square Root Sum of Squares (SRSS) combination method was used.

*5.5 Computer Modelling of the Frame*

All the analyses were made using the program Ruaumoko (Carr 2003).

For all beam and column members a Giberson model was used and the stiffness of columns was computed at each time-step using its current axial force.

For the non-linear time-history analyses, the small displacements analysis was used when P-delta effects were not considered and the large displacements analysis option when they were considered. In a large displacement analysis, the coordinates of every node, as well as the stiffnesses of all members, are updated at each time step allowing for geometry changes (Carr 2003).

*5.6 Hysteretic Models*

*5.6.1 Takeda Hysteretic Model*

In order to simulate the behaviour of each member, modified Takeda type hysteretic models were used in all elements of the frame. Figure 5.2 shows the parameters needed to define a Takeda type cycle used in the program Ruaumoko (Carr 2003).

To define the $\alpha$ and $\beta$ values used in the Takeda hysteretic loop, four laboratory tests performed at Auckland University (Fenwick et al. 1981) were analysed, two which have a typical beam cross section and steel distribution and two with a typical column cross section and steel distribution subject to a low axial load. The α and β factors obtained were 0.35 and 0.43 respectively for beams and the corresponding values for columns were 0.23 and 0.60.

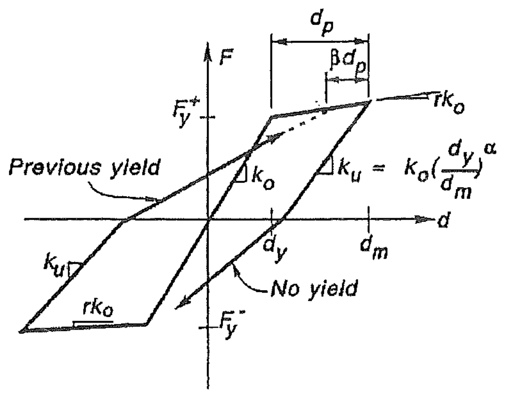



**Figure 5-2 Variables that define a Takeda type model**

*Figure taken from "Ruaumoko Manual ®" under authorisation of Carr, A. J.*

The factor controlling the post-yield slope of each member (r) was modified so that the post-yield stiffness obtained from the pushover analyses to the frame without P-delta effects, using a set of forces proportional to the distribution defined by NZS 4203:1992 (Standards New Zealand. 1992) for an equivalent static analysis, would be of the order of 3 % of the yield strength for every increment in ductility, as shown on figure 5.3. The value of 3% was selected so that when the structure reaches a ductility (μ) of 6, the total increment above the ductility one value is approximately equal to 15% of the base shear for a ductility of one. This increment of 15% is similar to that observed in studies where reinforced concrete members were tested (Fenwick et al. 1981; Gill 1979; Munro 1976). The same value of r was used for every frame with the same number of levels. The values of r used for each structure can be found in appendix A.

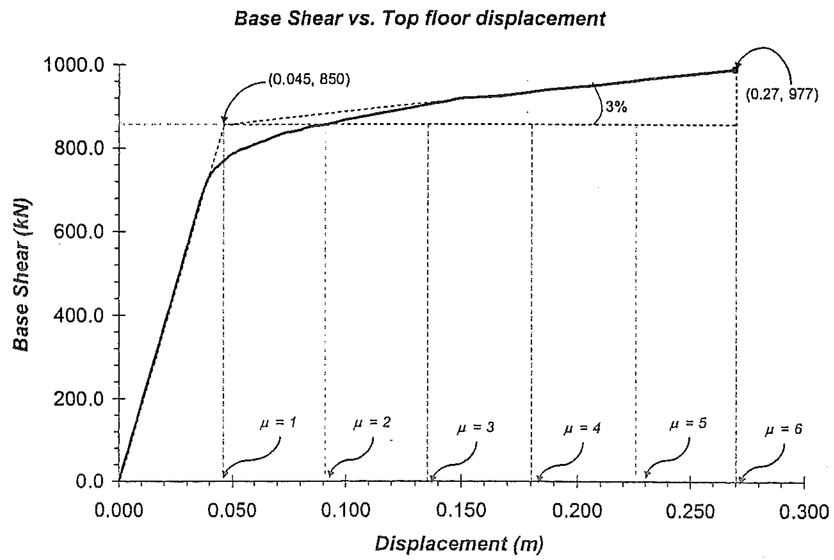
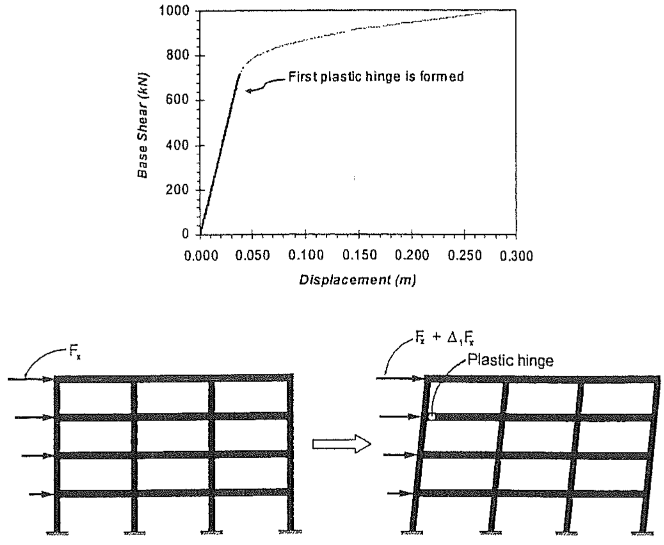



**Figure 5-3 Inelastic push-over analysis to a four storey structure**

(a)

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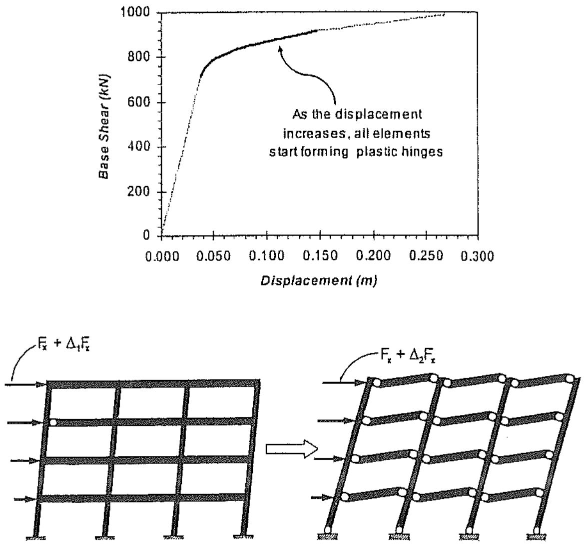



(c)

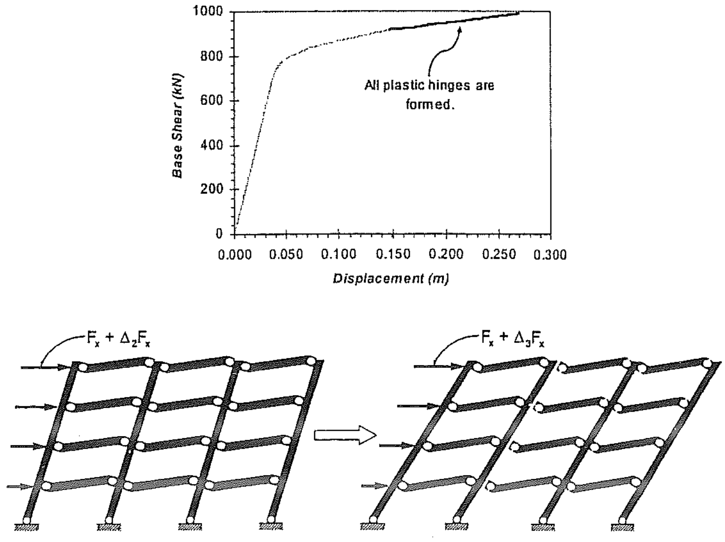



**Figure 5-4 Plastic hinge formation process in a pushover analysis**

Figure 5.4 shows each of the 3 stages (a, b and c) that a structure undergoes when an inelastic push-over analysis is performed. The graph shows the resulting base shear versus top floor displacement obtained from an inelastic push-over to a 4 storey structure.

The highlighted part of the line on the graph represents the stage that is being illustrated. Figure 5.4.a shows the initial stage of a push-over analysis when the structure remains elastic until the first plastic hinge appears in the structure. After the first hinge is formed, the stiffness of the structure starts decreasing. This decrement continues as more plastic hinges in other elements are formed. As shown in figure 5.4.b, at the end of this stage a mechanism is formed. With more displacement, the lateral resistance can only increase due to strain hardening of the reinforcement, as shown in figure 5.4.c. Because of this, in this project the post-yield stiffness of the structure was adjusted to represent approximately the same post-yield stiffness that would be resisted by the yielding elements.

### *5.6.2 Bilinear Hysteretic Model*

In some cases, a bilinear hysteretic model was used to see the effect this would have on the behaviour of the structure compared to the "same" structure using the modified Takeda model. The only factor needed to define the bilinear model is r. This was calculated as previously described. These analyses were made so that the significance of the two hysteretic models on P-delta effects could be assessed and so that the results can be related to previous studies (Bernal 1987; Fenwick et al. 1992; Kelly 1977; Moss and Carr 1980).

## *5.7 Modelling of Beam-Column Joints*

As illustrated in Figure 5.5, internal beam-column joint zones deform in shear. The practical problem is to allow for this deformation in a manner which can be readily adopted in an analysis. Because shear deformations are not modelled, the beam-column joint deformation is in this project incorporated through an approximation by modifying the curvature of the elements. In this study, three different approaches were examined before one of them was selected and used in the equivalent static, modal and time history analyses.

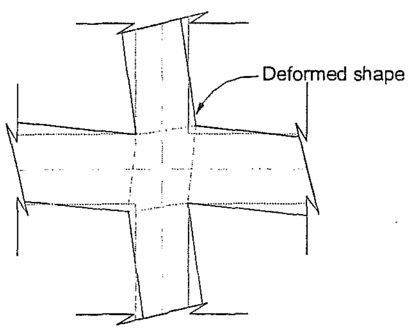
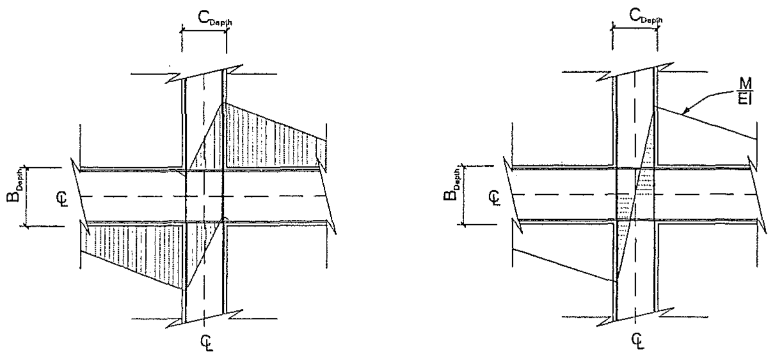



**Figure 5-5 Deformed shape of beam-column joint when subject to lateral forces**

Figure 5.6 shows how strain in the longitudinal reinforcement in a beam varies through a joint zone. From this figure it can be seen that flexural curvature is zero at the column centreline and it increases linearly to the column face. The flexural rotation between the two points can be approximately modelled by:

a) Doubling the flexural stiffness of the beam member at the joint zone as illustrated in figure 5.7.A;
b) Use of rigid members over part of the joint zone, as illustrated in Figure 5.7.B;
c) The use of a flexural spring at the beam and column faces as illustrated in Figure 5.7.C.




Strain distribution in longitudinal steel — Approximate curvature

**Figure 5-6 Strain diagrams and curvature at joint**

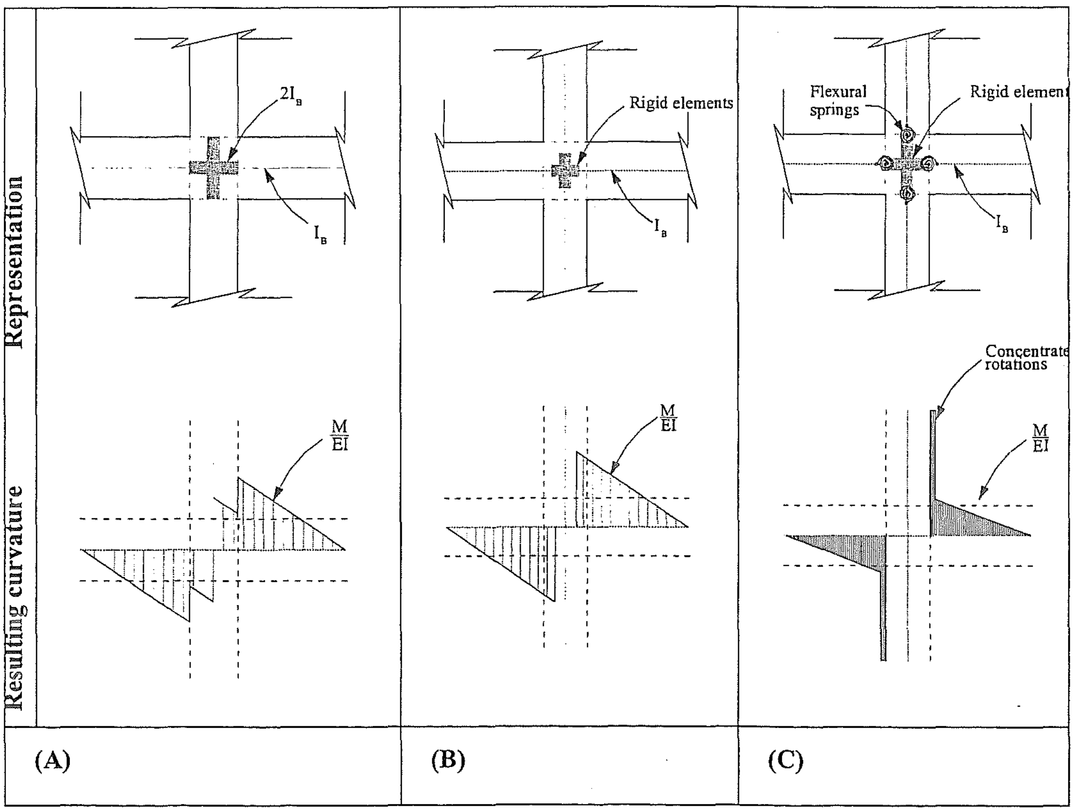


**Figure 5-7 Different options to model Beam-Column joints**

For the analyses in this work, the use of a flexural spring at the ends of columns and beams was adopted. Option A was discarded because the inclusion of very small elements at the beam-column joint area created problems when running the analyses due to the high stiffness ratio of elements in the model. Besides, the computational effort would have been much higher than the other two options. Option B was discarded because the beam and column actions were reported by the program at the end of the rigid element and not at column and beam face as required. When dynamic effects are present the variation of moments within the joint area can be considerable and they can not be easily extrapolated when inelastic actions are present.

In this work, the flexibility of the flexural springs for option C corresponds to that of the adjacent member (beam or column respectively) with a length of ¼ the length of the joint. The added flexibility can be calculated by the next expression:

$$f_{added} = \frac{L}{EI}$$

Where $f_{added}$ is the value of the added flexibility, L is the length of ¼ of the columns depth when modelling horizontal members and ¼ of the beams depth when modelling vertical members, E is the modulus of elasticity of concrete and I the effective moment of inertia of the adjacent section.

*5.8 P-Delta Effects*

The term P-delta refers to the additional actions by the axial force (P) when there is a horizontal displacement (delta) on a vertical element. This effect may reduce significantly the flexural capacity of a structure. The effect can be easily explained for a single degree of freedom system (SDOF) (Chung 1993).

Figure 5.8 represents a SDOF pinned at the base with an axial load P that could be an external load or the self-weight of the structure. If a horizontal displacement (Δ) is applied to the SDOF, it can be deduced from equilibrium that a force PΔ /H should be applied to maintain equilibrium.

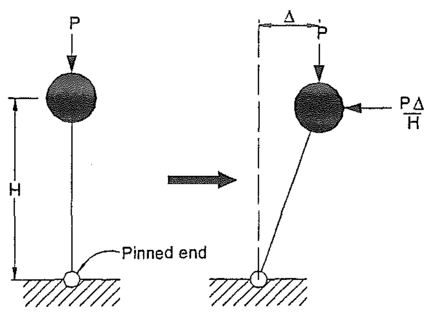



**Figure 5-8 P-delta effects on a SDOF structure**

If P-delta effects are considered, the necessary external force to induce a unitary displacement (stiffness) to a SDOF structure fixed at the base would be reduced on the same magnitude of the horizontal force induced by P-delta effects. This change in stiffness increases the fundamental period of the structure.

The change in stiffness due to P-delta effects for a SDOF structure can be calculated using the following equation:

$$k_1 = k_0 - \frac{P}{H}$$

where $k_1$ is the stiffness with P-delta effects, $k_0$ the stiffness without considering P-delta effects, P is the axial load and H the height. The change in the post-yield stiffness from P-delta effects can make the final slope to be negative as shown in figure 5.9. It should be noted that this negative slope is P-delta induced and is not coming from the material properties.

One of the most important characteristics of P-delta effects when a SDOF is subject to cyclic loading and inelastic behaviour is the tendency to yield in one direction only. Once a significant post-yield displacement takes place, the reduction of strength makes it more likely for the structure to sustain additional inelastic displacement in the same direction (Chung 1993). Because of this, the length of strong motions applied to the structure can have a considerable effect that may lead to incremental collapse. As illustrated in Figure 5.9, the lateral resistance is greater for the structure to return to its original position than to increase its displacement.

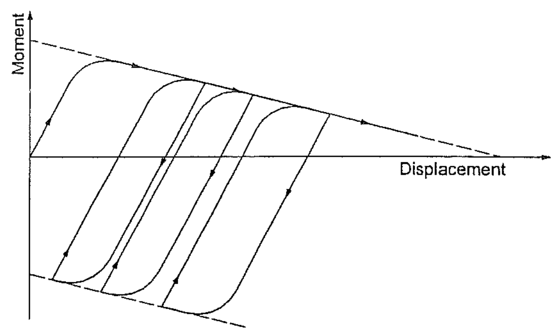



**Figure 5-9 Cyclic loading considering P-delta**

It has been shown (Chung et al. 1991; Fenwick et al. 1992; Zhao et al. 2001) that the most important factors affecting P-delta effects are the ductility, period of the structure and choice of hysteretic model.

For this work, the method recommended in the commentary to the New Zealand Loading Standard (Standards New Zealand. 1992) to asses P-delta effects in multi-storey buildings was used.

The steps to follow are:

1. Analyse the structure by the equivalent static or modal response spectrum method without considering P-delta actions.
2. Determine the maximum lateral deflection at each level and scale these values by the structure ductility factor to allow for inelastic deformation.
3. Assume all the gravity load is carried by columns which are pinned at each level and displaced as in 2 (see fig. 5.10). Find the lateral forces $F_{xi}$ required at each level for stability.
4. Calculate the value β, that makes an allowance for the ductility demand and K which makes an allowance for the period of the structure and the soil conditions. These values are defined by the following expressions, where μ is the ductility factor and $T_1$ is the fundamental period of the structure in seconds;

If $\mu \leq 3.5$ then $\beta = \frac{2\mu K}{3.5}$ and if $\mu > 3.5$ then $\beta = 2K$

For a rocky or very stiff soils and for intermediate soils,

$K = 1.0$; for $T_1 \leq 2.0$ seconds

$K = \frac{(6 - T_1)}{4}$; for $2.0 \leq T1 \leq 4.0$ seconds

$K = 0.5$; for $T1 > 4.0$ seconds

5. Forces found on step 3 are scaled by β. The scaled forces are applied to the structure and the deflections calculated.
6. The additional elastic P-delta deflections are multiplied by the structural ductility factor μ to get the inelastic deflection and added to the deflections obtained at step 2. The resultant deflections should comply with the maximum allowable inter-storey drift limits.

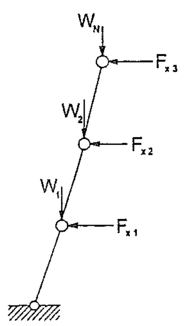



**Figure 5-10 Model to assess lateral forces induced by P-delta actions**

### *5.9 Moment Redistribution*

The purpose of moment redistribution in beams of a moment resisting frame is to get a more efficient design while maintaining the equilibrium for vertical and horizontal forces. According to Paulay and Priestley (Paulay and Priestley 1992) the advantages of moment redistribution are:

1. Reducing the absolute maximum moment and increasing moments in non-critical regions. If possible, the positive and negative moments will be set to be the same resulting in a symmetrical arrangement of flexural reinforcement in beams;
2. Equalize moment requirements for beams on opposite sides of an interior column to avoid the need to terminate reinforcement steel at interior beam-column joints;
3. If the minimum possible positive moment capacity is used for design, it is easier to comply with the requirement of having at least 50% of the positive capacity to resist negative moments. This measure will ensure that the designed members can develop the inelastic rotation demand;
4. If capacity design principles are followed, the flexural capacity of columns in ductile moment resisting frames will be dictated in most cases by the flexural capacity of beams so that if the flexural capacity of beams is reduced by redistribution, excessive flexural capacity on columns can be avoided.

One method of redistribution, which satisfies equilibrium for both horizontal and vertical forces is described in *"Examples of concrete structural design to New Zealand Standard 3101"* (Bull et al. 1998). With NZS 3101:1995 (Standards New Zealand. 1995) redistribution of moment is limited. The bending moment in any span is not allowed to be changed by more than 30% of the maximum bending moment in the span being considered in any load combination.

As an example, the moment redistribution at the 3rd level of six storey frame model 6/6/E/T (see section 5.12 and appendix A for details of the structure) is described. Table 5.2 shows the beam moments at the centreline of columns coming from the different elastic analyses. The Spectral Modal analysis just considered the bending moments coming from the appropriate response spectra. In the table the bending moments due to gravity loads, response spectral modal analysis and P-delta actions are found and summed to obtain the proper load combination.

**Table 5-2 Beam moments at column centreline**

| **Load combination** | **A to B** *(kN m)* | | **B to C** *(kN m)* | | **C to D** *(kN m)* | |
|---|---|---|---|---|---|---|
| **Gravity loads** | 41.6 | 44.9 | 45.0 | 45.0 | 44.9 | 41.6 |

| | | | | | | |
|---|---|---|---|---|---|---|
| **Spectral Modal analysis** | -284.2 | 261.4 | -245.0 | 245.0 | -261.4 | 284.2 |
| **P-Δ** | -93.2 | 85.8 | -81.4 | 81.4 | -85.8 | 93.2 |

To get the design moments for beams, the corresponding values at column faces are calculated. This is easily achieved using the relation between shear forces and moments, as illustrated in figure 5.11. Moments at column faces are given in Table 5.3.

**Table 5-3 Beam moments at column face**

| Load combination | A to B (kN m) | | B to C (kN m) | | C to D (kN m) | |
|---|---|---|---|---|---|---|
| **Gravity loads** | 30.8 | 33.8 | 34.1 | 34.1 | 33.8 | 30.84 |
| **Spectral Modal analysis** | -256.8 | 236.2 | -221.4 | 221.4 | -236.2 | 256.8 |
| **P-Δ** | -84.23 | 77.53 | -73.5 | 73.5 | -77.5 | 84.23 |
| Σ = | -310.2 | 347.5 | -260.8 | 329.0 | -279.9 | 371.8 |

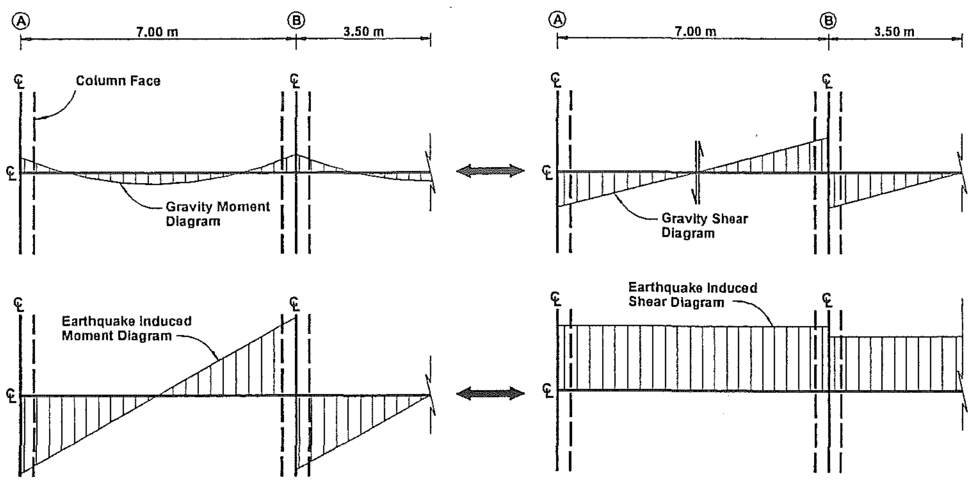



**Figure 5-11 Relation between moment and shear**

Now that the moments at column face are known the redistribution can be obtained by getting an average of the absolute moment at each column face for each span and then getting the average of these moments. This is:

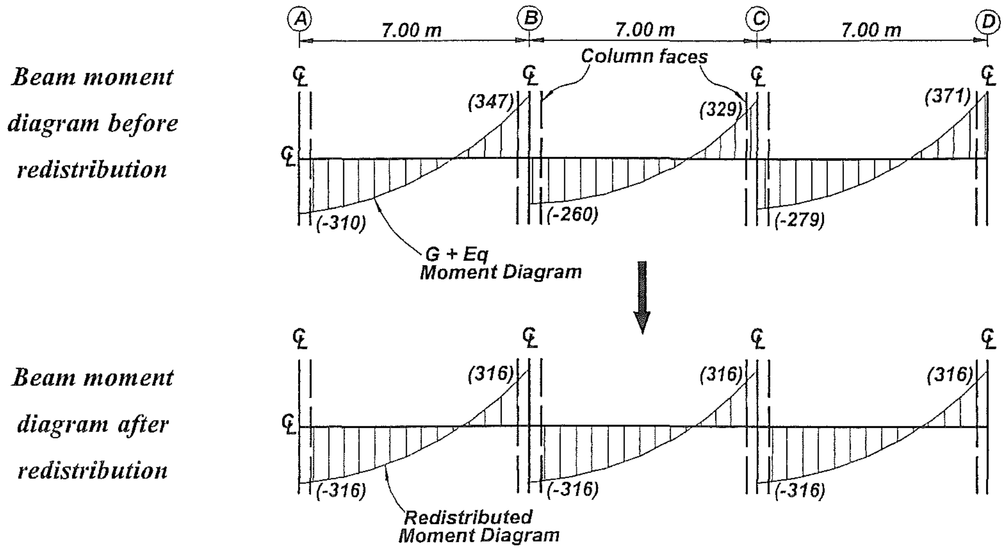
$$\text{Span A;} \quad \frac{310.2+347.5}{2} = \quad 328.8$$

$$\text{Span B;} \quad \frac{260.8+329.0}{2} = \quad 294.9$$

$$\text{Span C;} \quad \frac{279.9+371.8}{2} = \quad 325.8$$

$$\boxed{\textit{Average} = \phi M_n = \quad \mathbf{316.5}}$$

Given the regularity of the frame and the low axial load, the variation of the original moments to the redistributed moment is not too large. It can be verified that the redistribution value (316.5) complies with the 30% maximum moment redistribution. The redistribution process is illustrated on figure 5.12. The numbers between parentheses are the reduced moment $\phi$ $M_n$ at column face. For the time history non-linear analyses, the nominal capacity $M_n$ was used. $\phi$ was taken as 0.85. Further detail for the beam moment redistribution can be found elsewhere (Bull et al. 1998; Paulay 1980; Paulay 1988; Standards New Zealand. 1995).




**Note that moments are $\phi M_n$ moments at beam faces.*

**Figure 5-12 Beam moment redistribution**

**(All moments are in kN m)**

*5.10 Column Design Forces Assessment for a High Protection against Plastic Hinge Formation*

This procedure is that described in NZS 3101:1995 (Standards New Zealand. 1995). It applies to all columns except at ground level and the top storey where plastic hinge formation is allowed.

Once the beam design moments at column face are obtained through moment redistribution, an extrapolation to get the moment at column centrelines using the shear force sustained at the column face. Following the example given in section 5.9, the redistributed beam nominal moments, $M_n$ at column centrelines are shown in figure 5.13.

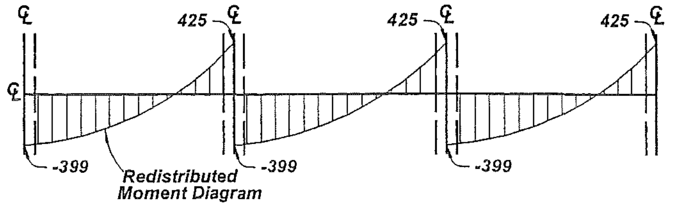



**Figure 5-13 Redistributed beam nominal moments at column centreline**

**(All moments are in kN m)**

To ensure a ductile failure mechanism forms in preference to other non-ductile failure modes, the structure is designed so that a weak beam-strong column mechanism is formed in the event of a major earthquake. To achieve this, the following steps are carried:

1. Determine the required design strengths of potential plastic hinge zones in the beams. Moment redistribution may be involved in this step.
2. Detail the beam potential plastic hinge zones for the required flexural strength.
3. Determine the flexural over-strength of the potential plastic hinge zones in the beams.
4. Determine the bending moments at the column centrelines due to beam over-strength and determine the beam over-strength input into the columns.
5. Get the equivalent static or first mode moments in the columns at beam centreline and scale them so that the sum of the absolute moments at each node is equal to the sum of beam over-strength moments at column centreline.
6. Scale the resultant column moments by the dynamic amplification factor, ω. (see section 5.10.1) .
7. Determine the bending moments at the beam face acting in the columns. This step involves interpolating the column bending moments at beam centreline in accordance to the expected shear forces.
8. Design columns at beam faces to satisfy the expected levels of axial loading and bending moment.

Figure 5.14 shows how the column bending moments at beam centreline coming from beam over-strength are obtained at level 3 from the example on section 5.9 at the interior joint on grid B where $M_{eq}$ is the sum of the absolute column moments at beam centreline coming from the equivalent static analysis, $M_{red}$ is the sum of the absolute redistributed beam nominal moments and $M_{top}$ and $M_{bot}$ are the resulting moments at beam centreline. To get the column design moments and shears, these moments should be further scaled by a corresponding dynamic amplification factor and an over-strength factor. These scaled moments at beam face are used to design the columns.

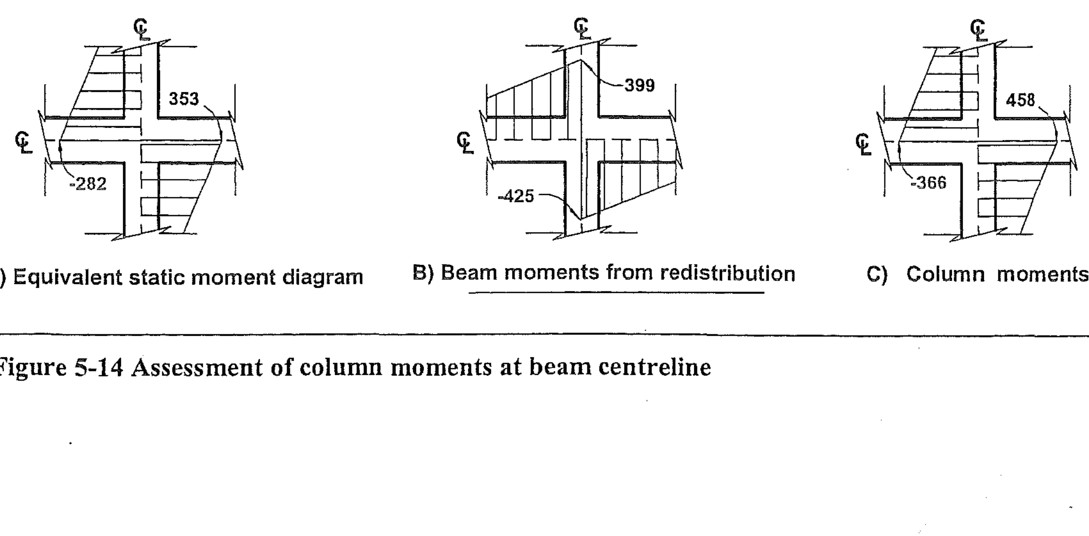
$$\frac{M_{abs}}{M_{eq}} = \frac{824}{635} = 1.30$$

$M_{eq} = 282 + 353 = 635$ kNm

$M_{red} = 399 + 425 = 824$ kNm

$M_{top} = -282 \times 1.30 = -366$ kNm

$M_{bot} = 353 \times 1.30 = 458$ kNm




A) Equivalent static moment diagram B) Beam moments from redistribution C) Column moments

**Figure 5-14 Assessment of column moments at beam centreline**

### *5.10.1 Dynamic Amplification and Over-strength Factor*

Recent studies made to 3, 4 and 8 storey frame buildings, show that a simple design approach based on the dynamic amplification factor modified accordingly to the ductility level would be appropriate but it was concluded that for frame structures further studies are required (Priestley 2003).

To define an appropriate design capacity for columns, a dynamic amplification factor, $\omega$ and an over-strength factor, $\phi_o$ are required. The former is needed mainly because as the structure is subject to the dynamic effects induced by an earthquake, the column moment pattern differs from that obtained from an elastic analysis. The main reason for this variation is higher mode effects which affect upper levels significantly. The over-strength factor allows for the strength variation that occurs in steel and concrete from their design values and for strain hardening effects, as shown in figure 5.15 a and b respectively. For design, a lower characteristic strength is used but to get an appropriate over-strength factor the upper characteristic strength must be considered.

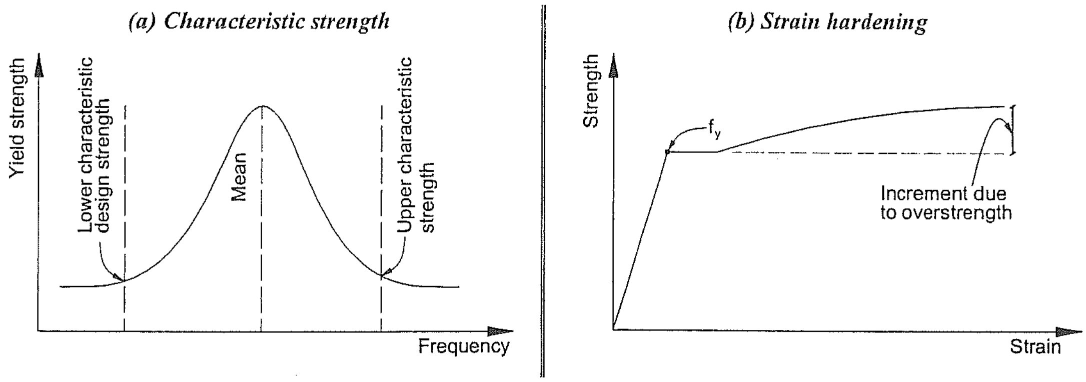



**Figure 5-15 Characteristic steel strength values**

Previous research undertaken at Canterbury University (Jury 1978; Paulay 1977) set the limits for these two values defined by the New Zealand Concrete Structures Standard (Standards New Zealand. 1995). Given the limitations of the time, the studies were made of a limited number of structures using a bilinear hysteretic model and neglecting P-delta effects.

When evaluating the dynamic amplification factor, concurrent lateral loading in both principal directions of the structure should be considered. Thus, when designing by NZS 3101:1995 (Standards New Zealand. 1995) to get the dynamic amplification factor, a distinction from one-way and two-way frames is stated. The dynamic amplification factor is given by the following expressions:

- For one-way frames

$$\omega = 0.6\, T_1 + 0.85;$$

but not less than 1.3 nor more than 1.8.

- For two-way frames

$$\omega = 0.5\ T_1 + 1.1$$

but not less than 1.5 nor more than 1.9.

This value for ω is used between 1/3rd of the height of the building and 2 levels below the top floor. At the base and the top floor, ω is taken as 1.0 or 1.1 for one or two-way frames respectively. Below 1/3rd of the height of the building, the value is linearly interpolated from the value taken at the base and that defined by the previous expressions at 1/3rd of the height. At the first level ω should not be less than 1.30 or 1.50 for one or two-way frames. At the level immediately below the roof level, ω should be taken as 1.30 or 1.50 for one or two-way frames respectively. When no point of contra-flexure in a storey is indicated by the elastic analysis, ω can be taken as the minimum for the first floor.

Research for a better assessment of the dynamic amplification factor in walls has been undertaken (Priestley and Amaris 2002) but it is suggested that further research is needed to account for the dynamic amplification factors in ductile moment resisting frames to get a more reliable response when structures are subject to dynamic actions such as earthquakes (Priestley 2003).

New Zealand Standard NZS 3101:1995 (Standards New Zealand. 1995) suggests an over-strength value of 1.25. This accounts for an increase of strength due to strain hardening and the possible scatter of the design and real strength of steel reinforcement. ***For this project an over-strength factor of 1.1*** was used given that all members were modelled using their nominal strength and certainty in the real strength was assumed.

### *5.10.2 Design Shear*

The Concrete Structures Standard NZS 3101:1995 (Standards New Zealand. 1995) says that in upper storeys, except at the top level where hinging in columns is allowed, the design shear for one-way frames should be calculated according to the following expression:

$$V^*_{col} = 1.3\ \phi_o\ V_E$$

where $V^*_{col}$ is the column design shear force, $V_E$ is the column shear force derived from lateral earthquake design forces and $\phi_o$ is the beam over-strength factor.

When designing the column at the base or at the top floor, a different expression applies given that hinging is allowed at these locations.

### *5.10.3 Design Axial Forces*

The design axial forces are calculated adding to the appropriate gravity load value the cumulative shear induced by the beams above the column section being considered. In the NZS 3101:1995 (Standards New Zealand. 1995) allowance is made to the low probability of all beams hinging at the same moment in time and thus a reduction factor, $R_v$ is introduced for the component of shear in the beams due to seismic actions.

### *5.10.4 Design Moments*

The design nominal moment as defined by NZS 3101:1995 (Standards New Zealand. 1995) is given by:

$$M^*_{col} = \phi_0\ \omega M_E - 0.3\ h_B\ V^*_{col}$$

where $\phi_0\ \omega\ M_E$ is the scaled moment coming from beam over-strength and scaled by the dynamic amplification factor and $0.3\ h_B\ V^*_{col}$ represent the difference between the column moment at beam centreline and that at beam face. Given that the design shear would generate an upper bound gradient when integrated to get the moment diagram, a 60% of the design shear $V^*_{col}$, is used to calculate the design moment at the beam face so that the moment reduction from the centreline to the beam face is given by:

$$M_{chg} = 0.6\ V^*_{col}\ 0.5\ h_b = 0.3\ h_B\ V^*_{col}$$

where $M_{chg}$ is the change in moment from the beam centreline to beam face, $V^*_{col}$ is the design column shear and $h_b$ is the depth of the beam. This is illustrated in figure 5.16.

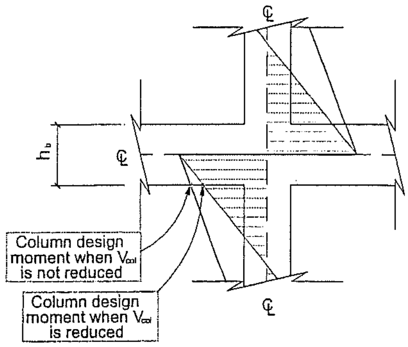



**Figure 5-16 Column moment at beam face**

As previously noted, this approach is valid for columns where plastic hinging is not desired. Special considerations should be taken for the top storey columns and the column section at the base.

### *5.11 Column Design Forces Assessment for a Low Protection against Plastic Hinge Formation Away from the Base*

As described in the draft for NZS 1170.5, to ensure that an appropriate failure mechanism is formed, the columns shall be proportioned so that the storey shear strength corresponding to a column-sway mechanism is equal to or greater than the calculated beam-sway storey shear strength times a dynamic amplification factor. To assess the beam-sway storey shear, the over-strength bending moments sustained at plastic hinge zones and the applied lateral loads should be considered. This procedure only applies if a point of contra-flexure is indicated in the columns in an equivalent static analysis. To calculate the storey beam-sway shear, the point of contra-flexure is assumed to be at half the storey height.

Figure 5.17 illustrates how the storey shear coming from beam over-strength bending moments and that coming from external lateral loads is distributed at the assumed inflexion point of columns to maintain equilibrium.

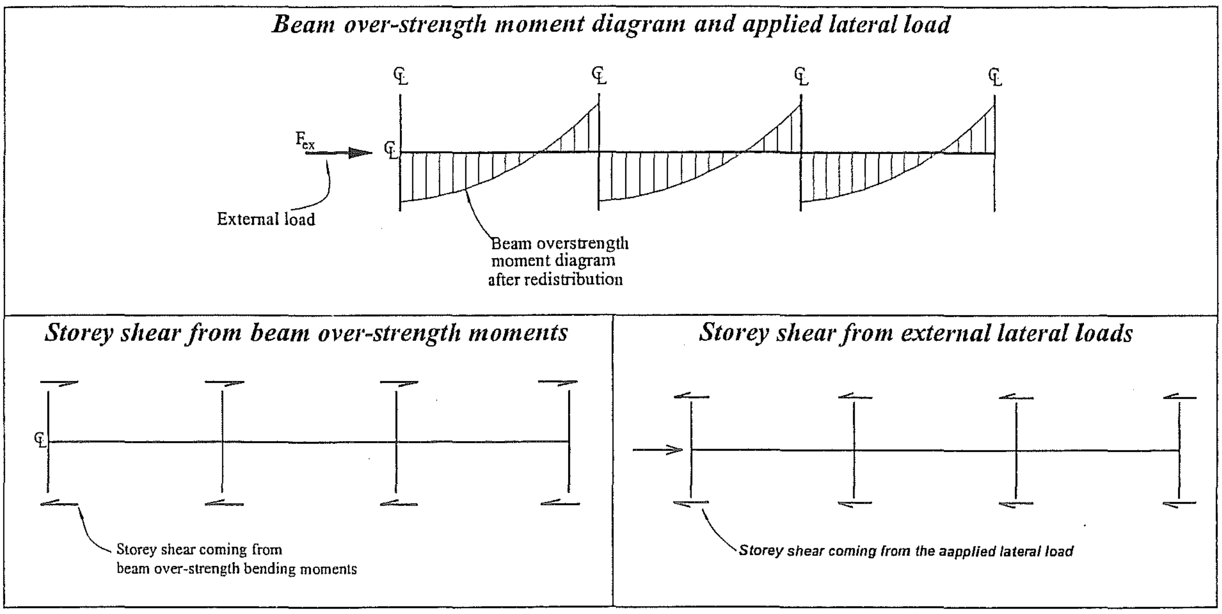
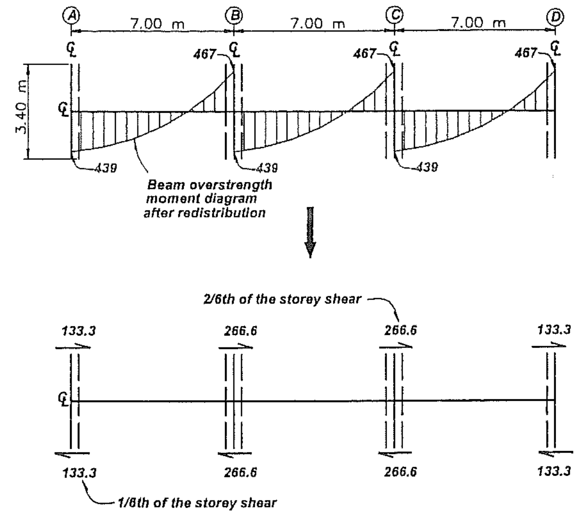



**Figure 5-17 Total storey shear components**

Taking again the third level of the 6 storey structure as an example, using the beam over-strength moments obtained in section 5.9, the storey shear induced by beam over-strength moments is calculated in figure 5.18.a. To achieve equilibrium, the lateral force from the equivalent static or first mode at the level being considered ($F_E$) is scaled so that the sum of the moments that it generates at beam centreline are equal to those at over-strength as shown in figure 5.18.b, where $F_E$ is the equivalent static lateral force at that level, $\Sigma M_E$ is the sum of the equivalent static beam moments at column centreline, $\Sigma M_o$ is the sum of beam over-strength moments at column centreline and $F_{ex}$ is the scaled equivalent static lateral force. These earthquake induced shears are added to those coming from the over-strength actions of beams and multiplied by the dynamic amplification factor ($\omega$) to get the total shear ($V_{tot}$ in figure 5.18.c). Since plastic behaviour is permitted at the columns, the total storey shear can be distributed throughout the columns in any way as long as equilibrium is maintained. The column design moments are estimated using the distributed shears applied at the inflexion point of each storey. The resultant column flexural strengths must in all cases be sufficient to sustain the over-strength moments in the beams. For this project, $1/6^{th}$ of the total shear was applied to each exterior column and $1/3^{rd}$ to each interior column. This allows for interior columns to hinge first. The shear at each column applied at the inflexion point will determine the column design moment at beam face as shown in figure 5.18.c.

(a)

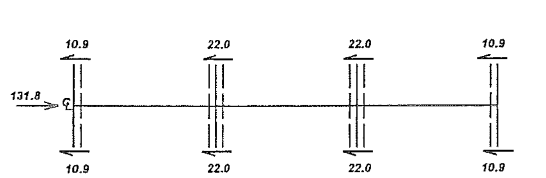



*Moments indicated at column centreline in kN m. Shears in kN

***Beam-sway storey shear from beam over-strength moments :***

$$V_{storey} = \frac{3(467)+3(439)}{3.4} = 800kN$$

(b)




*Moments indicated at column centreline in kNm. Shears in kN

***Scaling of equivalent static lateral force:***

$F_E = 96.2\ kN$

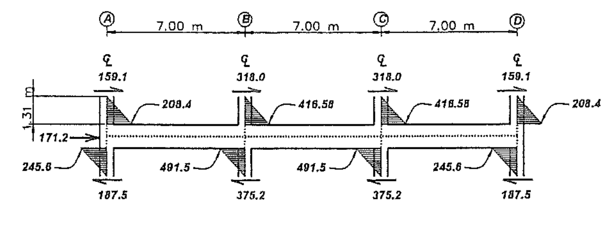
$\Sigma M_E = 2(353 + 327 + 309) = 1978\ kNm$

$\Sigma M_o = 3(467 + 439) = 2718\ kNm$

$$\Sigma M_o / \Sigma M_E = 1.37$$

$$F_{ex} = F_E x \, \Sigma M_o / \Sigma M_E = 131.8kN$$

(c)




*Moments indicated at beam face in kNm. Shears in kN

***Total shear that needs to be resisted by columns:***

$\omega = 1.3$

$V_{A\ upper} = 1.3\ x\ (133.2 - 10.9) = 159.1\ kN$

$V_{A\ lower} = 1.3\ x\ (133.3 + 10.9) = 187.5\ kN$

$V_{B\ upper} = 1.3\ x\ (266.6 - 22.0) = 318.0\ kN$

$V_{B\ lower} = 1.3\ x\ (266.6 + 22.0) = 375.2\ kN$

**Figure 5-18 Column design actions obtained by the NZS 1170.5 draft**

## *5.12 Frame Properties*

A part of a 150 mm deep slab was considered to contribute to the second moment of area of beams, as described by NZS 3101:1995 (Standards New Zealand. 1995) and illustrated in figure 5.19. To calculate the effective moment of inertia ($I_{effective}$), the beam gross moment of inertia was multiplied by a factor of 0.4 to account for the cracking of concrete. Columns were modelled with a rectangular cross section. The effective moment of inertia ($I_{effective}$) for columns was calculated by multiplying its gross moment of inertia by a factor of 0.4. The effective moment of inertia of all members for all sections is listed in Appendix A.

The design actions were found from a modal analysis and an analysis for P-delta effects as outlined in section 5.8. The bending moments were redistributed as outlined in section 5.9. The required nominal strength at the base of ground level columns were calculated by adding the modal and P-delta actions and then dividing the sum by the strength reduction factor which was 0.85. A summary of the section properties is given in Table 5.4. More details of the design of each structure can be found in Appendix A.

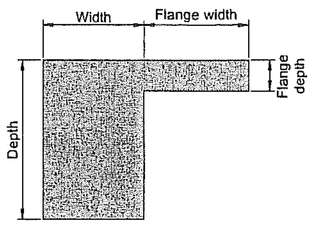



**Figure 5-19 Beam cross section**

**Table 5-4 Section properties for different buildings**

| | *Beams** | | *Columns* |
|---|---|---|---|
| | *Section (mm x mm)* | *Flange width** (mm)* | *Section (mm x mm)* |
| *Four storey building* | 700 x 400 | 580 | 625 x 500 |
| *Six storey building* | 775 x 450 | 580 | 675 x 600 |
| *Twelve storey building A* | 900 x 550 | 580 | 750 x 650 |
| *Twelve storey building B**** | *Lower:* 900 x 550<br>*Upper:* 750 x 550 | *Lower:* 580<br>*Upper:* 580 | *Lower:* 775 x 650<br>*Upper:* 700 x 600 |

**The flange depth is 150 mm in all beams.*

*** As described by NZS 3101:1995, just half the width of the flange is used to calculate $I_{effective}$.*

****The lower 6 storeys have a different beam and column section than the upper 6 storeys*

Each of the designed buildings is identified according to the following notation:

*No. of storeys/Ductility factor/Type of columns/Hysteretic model*

where type of columns can be:

- **E:** for elastic columns except at the base
- **3101:** Columns designed according to NZS 3101:1995 (Standards New Zealand. 1995) and
- **1170:** Columns designed for limited protection against plastic hinges according to NZS 1170.5 draft.

and the hysteretic models can be:

- **B:** Bilinear hysteretic model (see section 5.6)

- **T:** Takeda type hysteretic model (see section 5.6)

so that for example, building 6/6/E/T is a 6 storey building designed using a structural ductility factor of 6 and when inelastic analyses were performed, elastic columns were used except at the base and a Takeda hysteretic model controlled the behaviour of the inelastic elements.

**Table 5-5 Properties of different buildings**

| **Four storey building** | **Ductility factor (μ)** | **Beams Comply with minimum steel requirements** | **Column Design method** | **Hysteretic model** |
|---|---|---|---|---|
| 4/6/E/T | 6 | Yes | Elastic columns | Takeda |
| 4/6/3101/T | 6 | Yes | NZS 3101:1995 | Takeda |
| 4/6/1170/T | 6 | Yes | NZS 1170.5 | Takeda |

| **Six Storey building** | **Ductility factor (μ)** | **Beams Comply with minimum steel requirements** | **Column Design method** | **Hysteretic model** |
|---|---|---|---|---|
| 6/6/E/T* | 6 | No | Elastic columns | Takeda |
| 6/6/E/T | 6 | Yes | Elastic columns | Takeda |
| 6/6/3101/T | 6 | Yes | NZS 3101:1995 | Takeda |
| 6/6/1170/T | 6 | Yes | NZS 1170.5 | Takeda |
| 6/6/E/B | 6 | Yes | Elastic columns | Bilinear |
| 6/4/E/T | 4 | Yes | Elastic columns | Takeda |
| 6/2/E/T | 2 | Yes | Elastic columns | Takeda |
| 6/1/E/T | 1 | Yes | Elastic columns | Takeda |

| **Twelve Storey building A** | **Ductility factor (μ)** | **Beams Comply with minimum steel requirements** | **Column Design method** | **Hysteretic model** |
|---|---|---|---|---|
| 12A/6/E/T | 6 | Yes | Elastic columns | Takeda |
| 12A/6/3101/T | 6 | Yes | NZS 3101:1995 | Takeda |
| 12A/6/1170/T | 6 | Yes | NZS 1170.5 | Takeda |
| 12A/6/E/B | 6 | Yes | Elastic columns | Bilinear |

| *Twelve storey building B* | **Ductility factor (μ)** | **Beams Comply with minimum steel requirements** | **Column Design method** | **Hysteretic model** |
|---|---|---|---|---|
| 12B/6/E/T | 6 | Yes | Elastic columns | Takeda |
| 12B/6/3101/T | 6 | Yes | NZS 3101:1995 | Takeda |
| 12B/6/1170/T | 6 | Yes | NZS 1170.5 | Takeda |
| ****Note that this building is the only one where beams do not comply with minimum steel requirements set by NZS 4203:1992. It will be referred to as 6/6/E/T**** | | | | |

The seismic weight acting with each level was 2,200 kN. A small portion of this weight, which consisted of the self-weight of the frame, was considered as inducing axial loads in the columns. In order to model the additional mass contributing to the inertial effects, an extra column with all its members pinned at each level, was assumed to support the rest of the gravity load. Pin-ended horizontal members connected the pin-ended column members at each level to the frame. When one or more of the node points in the pin-ended column is deflected, the change in direction of the axial load induces a lateral force in the pin-ended horizontal member. This lateral force, which represents the P-delta actions, is transmitted to the frame. Figure 5.20 illustrates this process.

The Ruaumoko (Carr 2003) input file for the non-linear time-history analysis for structures 4/6/3101/T, 6/6/3101/T, 12A/6/3101/T and 12B/6/3101/T can be found on Appendix F. In all elements when subject to plastic behaviour, plastic hinges were modelled with a length of half the depth of the member.

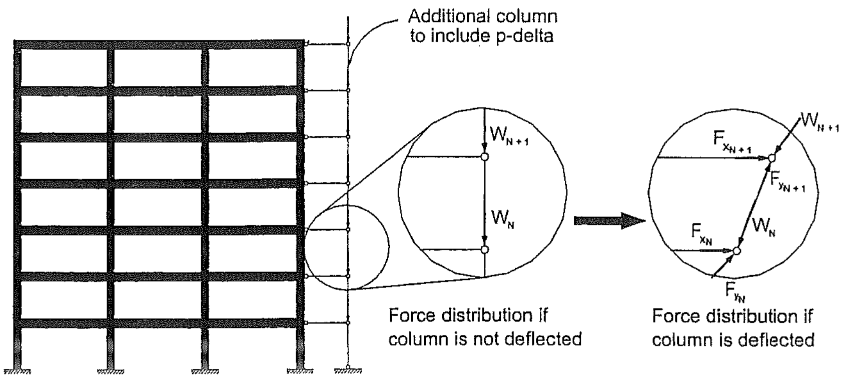



**Figure 5-20 Graphic representation of model with an additional column to consider P-delta effects**

## 5.13 Non-linear Time-history Analyses

### 5.13.1 Parameters

For the non-linear time-history analyses the Newmark constant average acceleration integration method was used. The damping was modelled as 5% in all modes. A time step of 0.005 seconds was used in most cases but for some analyses it needed to be reduced.

### *5.13.2 Return period 475 years ground acceleration records*

Eight ground acceleration records were used. The first four, shown in figure 5.21, were modified from its original form so that their acceleration response spectra would fit the 5% damped acceleration response spectra for intermediate soils and a ductility of 1 defined by NZS 4203:1992 (Standards New Zealand. 1992). Ground acceleration records 1 and 2 were originated by the "1940 El Centro" earthquake using the components North-South and East-West respectively. Records 3 and 4 were recorded at Kern County, California in 1952 at TAFT station. These records will be referred to as Modified El Centro NS, Modified El Centro EW, Modified TAFT1 and Modified TAFT2 respectively.

The use of these modified records is believed to be conservative as the response spectrum is at the design level over the whole spectrum, while in practice earthquake records would only reach the design level over a restricted portion of the design spectrum. An extra input of energy is expected to be induced affecting more significant higher mode response. Using records scaled in this manner was expected to have the advantage of reducing scatter of results due to the less jagged nature of their response spectra.

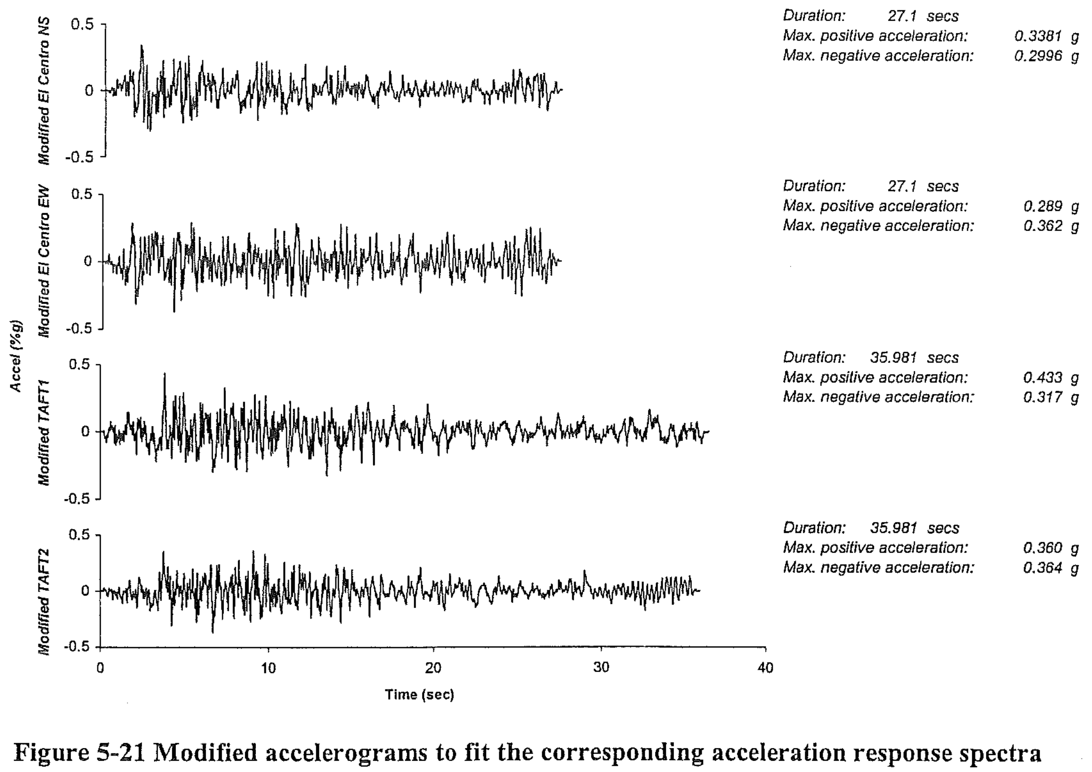



**Figure 5-21 Modified accelerograms to fit the corresponding acceleration response spectra**

In figure 5.22 the acceleration response spectra for the four selected records is compared to the design spectra defined by the loadings standard (Standards New Zealand. 1992). It should be noted that these records were further scaled to the zone factor, Z. The structural performance factor used for the elastic equivalent static and modal analyses was left out for the non-linear time-history analyses.

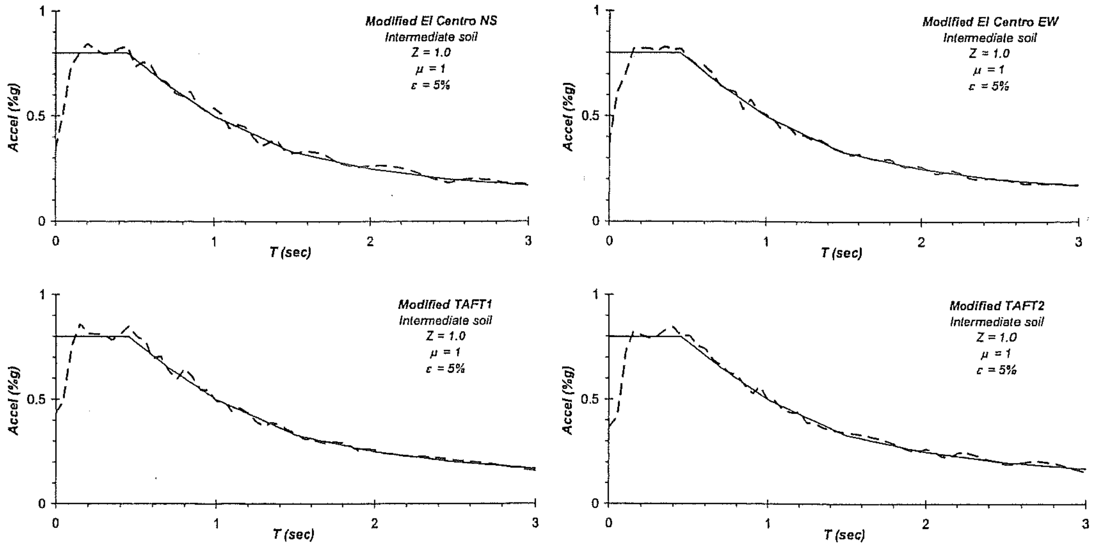



**Figure 5-22 Acceleration response spectra generated by the modified accelerograms and compared to the design response spectra defined by NZS 4203:1992**

The second set of records were used as originally recorded, but scaled linearly by a factor $k_1$ to obtain the closest fit to the design acceleration response spectra on a period ranging from $0.4T_1$ to $1.5T_1$, where $T_1$ is the fundamental period of the structure, using the least of squares method as outlined by the draft NZS 1170.5. Hence, each structure would have a $k_1$ scaling factor in accordance to its fundamental period. The factor $k_2$ or family scaling factor was not applied. The family scaling factor is calculated so that at least one of the records has all its ordinates above the design acceleration response spectra in the period range of interest. This was thought to be unrealistic, conservative and that would produce responses which are not representative of a 475 years return period event.

The records selected are two components for the 1940 El Centro earthquake and 2 components for the 1978 Tabas, Iran earthquake and can be seen in figure 5.23. These records will be referred to as El Centro 1, El Centro 2, Tabas 1 and Tabas 2 respectively.

It should be noted that these natural records were scaled to the acceleration response spectra defined in the NZS 1170.5 draft for shallow soil, which is similar to the intermediate soil spectra for NZS 4203:1992 (Standards New Zealand. 1992). The comparison of these two design spectra for the Wellington area is shown in figure 5.24.

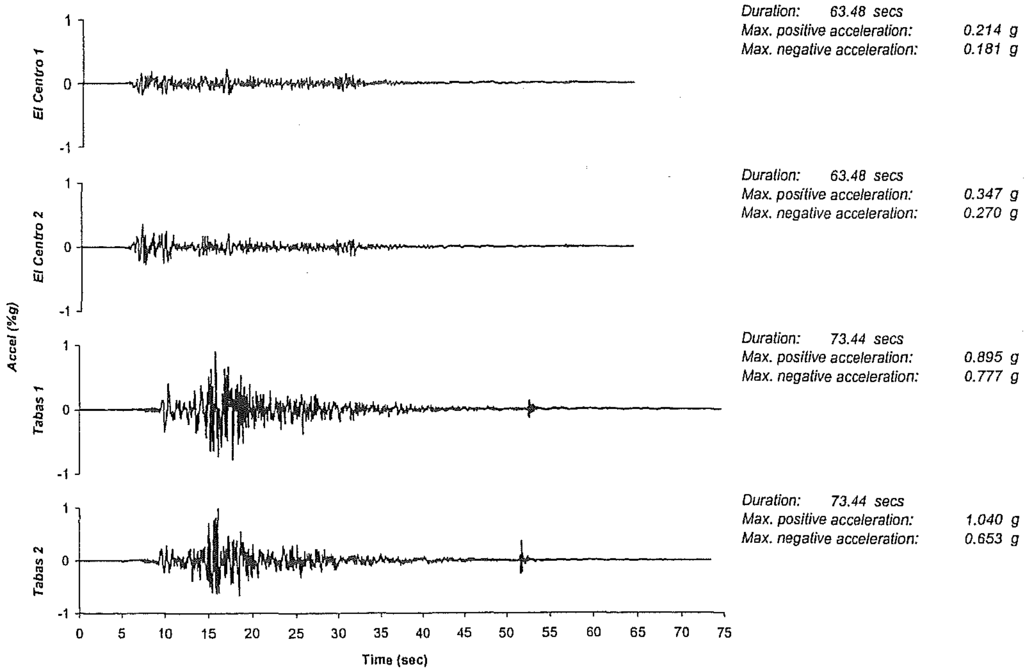



Figure 5-23 Set of original ground acceleration records

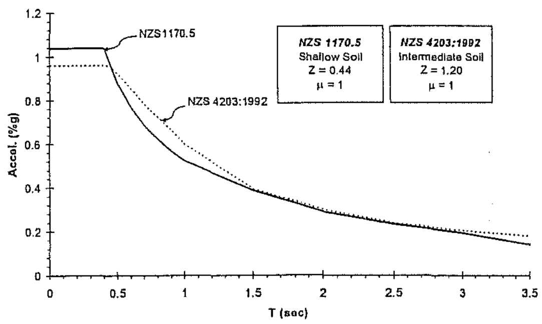



**Figure 5-24 Comparison of design response spectra**

Table 5.6 shows the scale factors used for each of the natural records for each of the structures that were analysed. Figure 5.25 shows the scaled response spectra for each of the structures for the natural records. In each graph the range period of interest is delimited by three vertical dotted lines. The centre vertical dotted line represents the fundamental period of the structure. The design response spectra is already scaled by the appropriate zone factor.

**Table 5-6 Scaling factors for each structure for each of the natural records**

| | *$k_1$ scaling factors* | | | |
|---|---|---|---|---|
| | *El Centro 1* | *El Centro 2* | *Tabas 1* | *Tabas 2* |
| *4 Storey* | *1.55* | *1.14* | *0.54* | *0.61* |
| *6 Storey* | *1.63* | *1.28* | *0.55* | *0.66* |
| *12 Storey A* | *1.61* | *1.43* | *0.54* | *0.77* |
| *12 Storey B* | *1.59* | *1.44* | *0.56* | *0.78* |

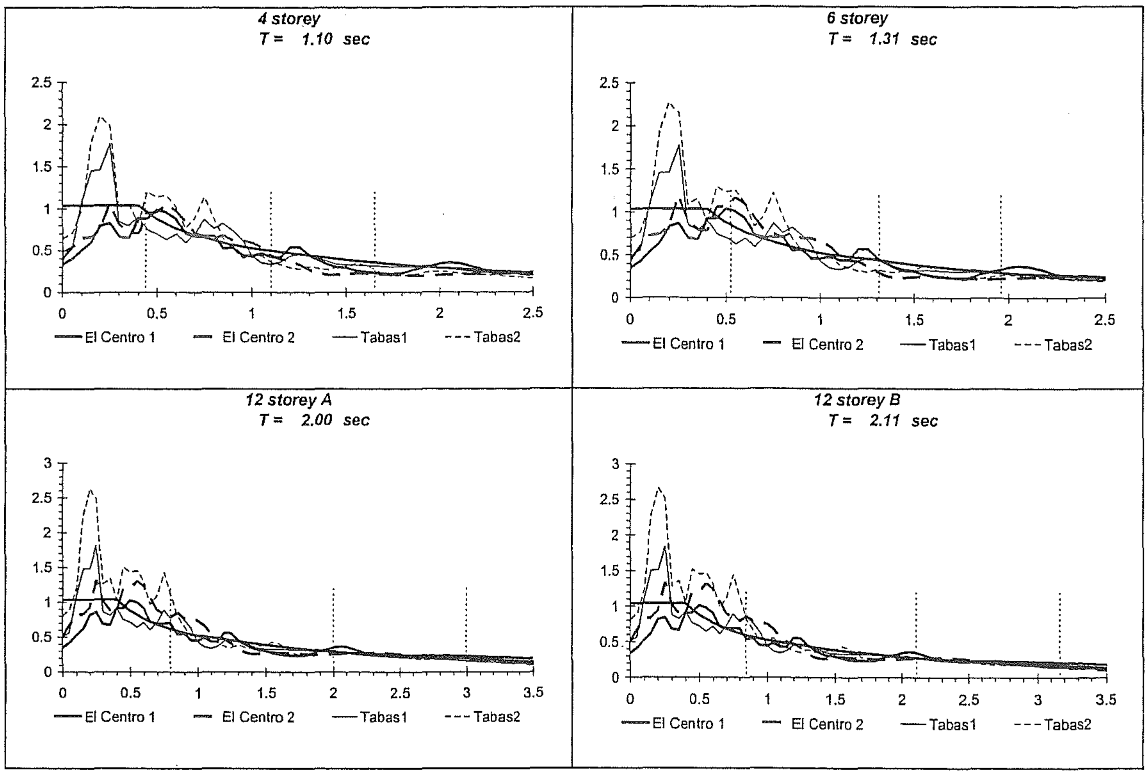



**Figure 5-25 Scaled acceleration response spectra for natural records**

### *5.13.3 Return period 2,500 years ground acceleration records*

The natural ground acceleration records representing the 475 year return period earthquake were found suitable to represent a 2,500 year return period earthquake on the Wellington area when they are scaled by a factor of 1.8[1]. The two natural ground acceleration records which show the most critical structural response will be scaled and used to represent 2,500 years return period earthquakes

[1] *The selection of the records representing a 2,500 years return period earthquake was made under the advice from Doctor G. McVerry from the New Zealand Institute of Geological and Nuclear Sciences. At the time this work was being written, studies were being made by Jian Zhang to adjust some records obtained in Japan that would be better examples to represent a large subduction zone such as that in Wellington but given time limitations they could not be included in this work*

# 6. SIX STOREY BUILDING

The maximum allowable inter-storey drift according to the Loadings Standard 4203:1992 (Standards New Zealand. 1992) when designed using the equivalent static or modal methods of analysis, for a total height of the frame of 20.4 m, is 1.82%. The corresponding limit for non-linear time-history analyses is 2.5%.

## *6.1 Equivalent Static Analysis Results*

The fundamental period estimated using the Rayleigh method was 1.31 seconds. The calculated base shear for the equivalent static analysis was 732.2 kN and the calculated maximum inter-storey drift for the equivalent static analysis after adding P-delta effects as recommended by the commentary for the Loadings Standard (Standards New Zealand. 1992) was 1.82%. Columns two and three in Table 6.1 show the resulting forces and inter-storey drifts from the equivalent static analysis respectively. The inter-storey drifts were reduced, as detailed in clause 4.8.1.5 of the Loadings Standard, NZS 4203:1992 (Standards New Zealand. 1992). This reduction is made as the equivalent static method over-estimates the seismic displacements compared with those of the equivalent results allowed by a modal analysis. The resulting deflection was then scaled by the structural ductility factor to give the resultant inter-storey drift in column 3. It can be seen that because P-delta effects are included, a considerable increase in the global stiffness of the structure is needed to comply with the maximum permitted inter-storey drift requirements. If P-delta effects are not considered, the maximum inter-storey drift coming from the equivalent static analysis would be 1.27%, which is well below the allowable 1.82% for this structure.

**Table 6-1 Equivalent static analysis results for the six storey frame**

| *Storey* | *Eq static Forces (kN)* | *Eq. Static Inter-storey drifts (%)* | *P-delta Forces* (kN)* | *Eq static plus P-delta Inter-storey drifts (%)* |
|---|---|---|---|---|
| *1* | 32.1 | 0.97 | -23.5 | 1.37 |
| *2* | 64.2 | 1.27 | 69.2 | 1.82 |
| *3* | 96.2 | 1.19 | 75.2 | 1.65 |
| *4* | 128.3 | 1.01 | 65.7 | 1.32 |
| *5* | 160.4 | 0.78 | 47.2 | 0.94 |
| *6* | 251.1 | 0.48 | 21.1 | 0.58 |
| *$\Sigma$=* | *732.2* | | *254.9* | |

* *P-delta forces have been multiplied by $\beta$ factor.*

### *6.2 Modal Analysis Results*

The dynamic properties of the six storey frame are listed in Table 6.2. The square root sum of the squares (SRSS) modal combination method was used to find the design actions.

**Table 6-2 Dynamic properties of the 6 storey frame**

| *Mode* | *Effective mass (Tonnes)* | *% Mass (Cumulative)* | *Period (seconds)* | $V_{max}$ *(kN)* |
|---|---|---|---|---|
| *1* | *1128.0* | *84* | *1.31* | *593.3* |
| *2* | *133.1* | *94* | *0.42* | *198.5* |
| *3* | *48.6* | *97* | *0.24* | *72.6* |
| *4* | *23.1* | *99* | *0.16* | *34.5* |
| *5* | *10.3* | *100* | *0.12* | *15.4* |
| *6* | *2.7* | *100* | *0.10* | *4.1* |
| $\Sigma =$ | ***1345.8*** | | ***SRSS =*** | ***631.0*** |

Table 6.3 shows the modal combined inter-storey drifts, P-delta forces factorised by β and the total inter-storey drifts, which is the addition of combined modal and P-delta inter-storey drifts which are shown as a percentage of storey height. These drifts have been scaled for inelastic action as required in NZS 4203:1992 (Standards New Zealand. 1992).

**Table 6-3 Modal inter-storey drifts and P-delta forces of six storey frame**

| *Storey* | *Modal combined Inter-storey drifts (%)* | *P-delta Forces* (kN)* | *Combined modal plus P-delta Inter-storey drifts (%)* |
|---|---|---|---|
| *1* | 0.96 | -12.82 | 1.36 |
| *2* | 1.21 | 76.95 | 1.74 |
| *3* | 1.08 | 73.77 | 1.50 |
| *4* | 0.88 | 59.01 | 1.15 |
| *5* | 0.65 | 40.30 | 0.79 |
| *6* | 0.38 | 16.69 | 0.44 |
| | $\Sigma =$ | **253.9** | |

* *P-delta forces are already multiplied by $\beta$ factor.*

A minimum base shear of 2.4% of the total weight of the building should be considered when designing through a modal analysis according to NZS 4203:1992 (Standards New Zealand. 1992). The NZS 1170.5 draft increases this value so that for Wellington a minimum base shear of 3.2% of the total weight of the building should be used. The new value for the minimum base shear is given by the following expression:

$$V_{min} = \left(\frac{Z}{20} + 0.02\right) x 0.8 W_T$$

where $V_{min}$ is the minimum base shear, Z is the zone factor and $W_t$ is the total weight of the structure. The 0.8 factor applies only to regular buildings.

*6.3 Non-linear Time-history Analyses Results*

Two sets of analyses were made for the 6 storey frame. In the first set (structure 6/6/E/T*), the design beam strengths were made equal to the values given by the modal analysis and calculated P-delta actions. In the second set (structure 6/6/E/T), the beam strength was not allowed to fall below the strength corresponding to the minimum reinforcement ratio permitted in NZS 3101:1995 (Standards New Zealand. 1995). With this second set the strengths of beams at levels 3, 4, 5 and 6 were the same. Figures 6.1 and 6.2 show that the inter-storey drifts and maximum displacements are more critical for the 6/6/E/T frame than for the 6/6/E/T* frame when subject to the modified ground accelerations described earlier. In all the inter-storey drift graphics the units are in percentage of the storey height, the 2.5% inter-storey drift limit is marked by a vertical line and the $y$ axis indicates the storey and not the level as stated in all the graphs, at which this inter-storey drift is acting.

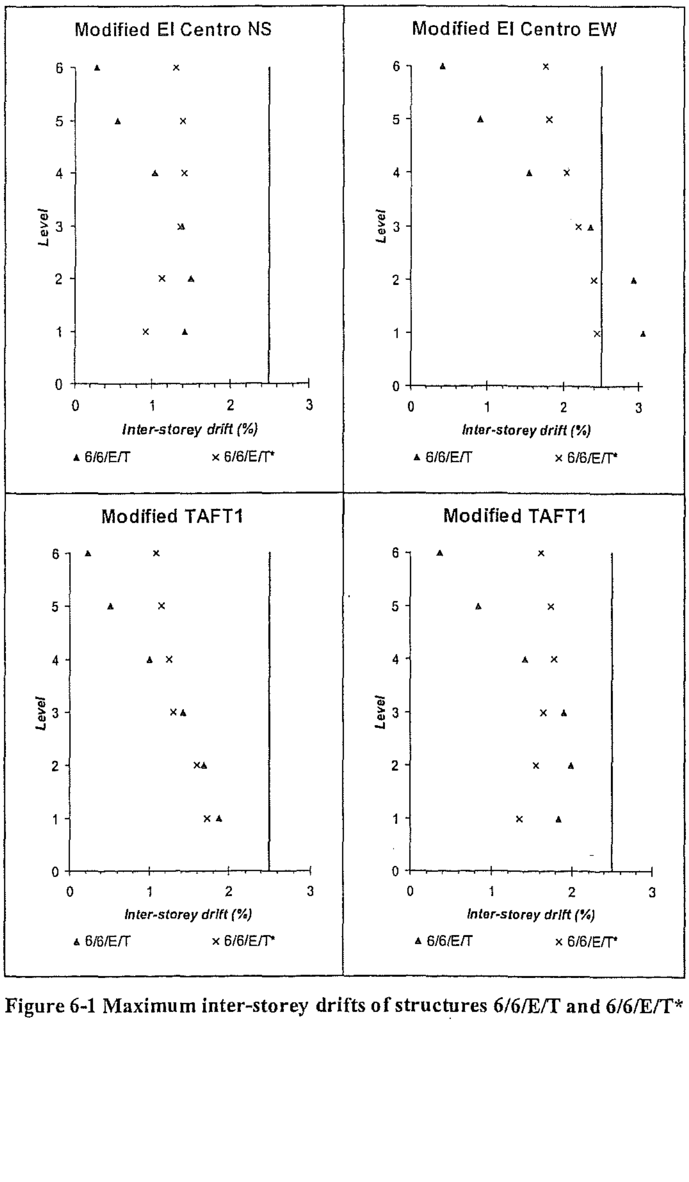



**Figure 6-1 Maximum inter-storey drifts of structures 6/6/E/T and 6/6/E/T***

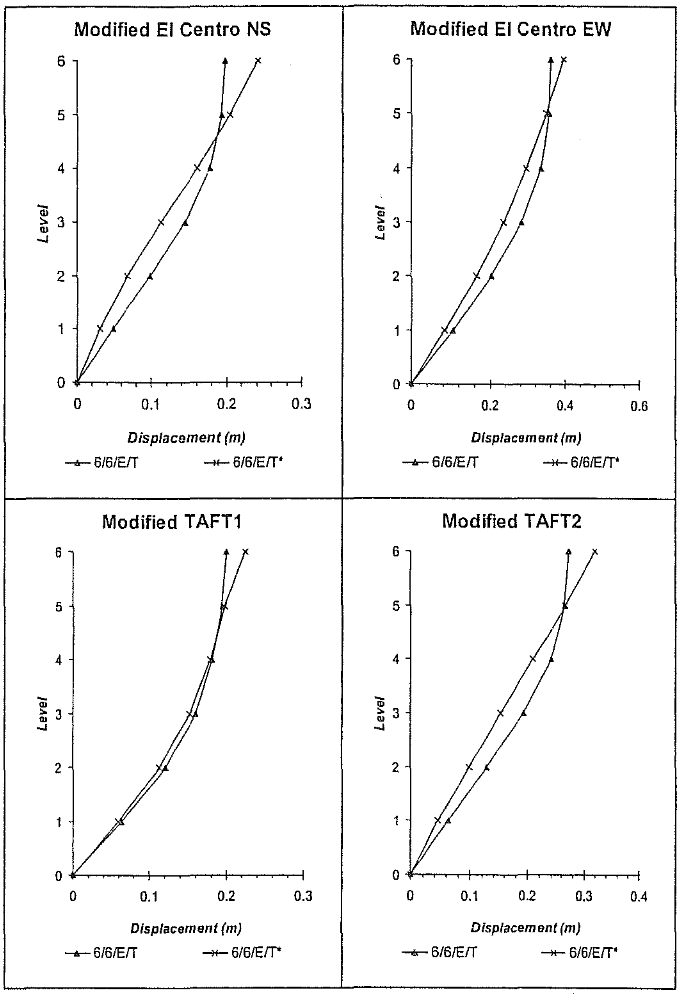
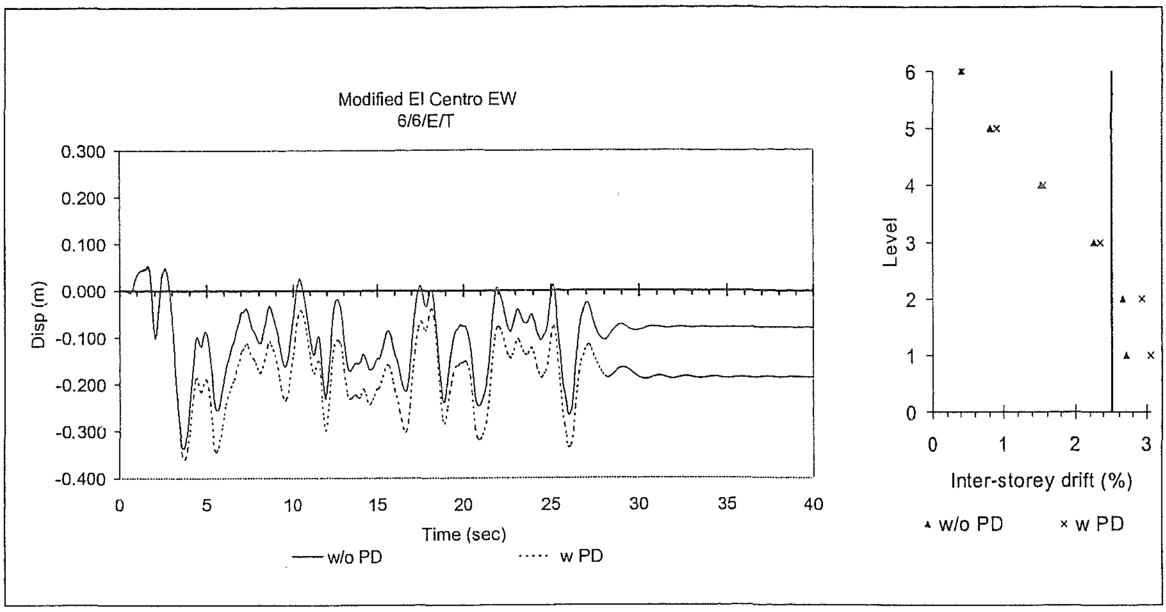



**Figure 6-2 Maximum displacements of structures 6/6/E/T and 6/6/E/T***

It can be seen that for model 6/6/E/T, the upper storeys did not contribute as much to the dissipation of energy as those for frame 6/6/E/T*. This is deducted from the small inter-storey drifts and the small variation on maximum displacement that the upper storeys have in structure 6/6/E/T.

Because real structures comply with minimum steel requirements and given that this is the more critical case, from here onwards the strength of the beams was not allowed to decrease below the value corresponding to the minimum reinforcement requirement. It should be noted that since one of the main objectives of this work is to assess the efficiency of different column design procedures, the minimum steel requirements were not considered in the design of the columns.

Figure 6.3 shows the effects that P-delta actions have on structure 6/6/E/T. In this figure, the top floor displacement versus time and the inter-storey drift envelope both with and without P-delta effects are shown for the non-linear time-history analyses using the Modified El Centro EW and Modified TAFT1 ground acceleration records.



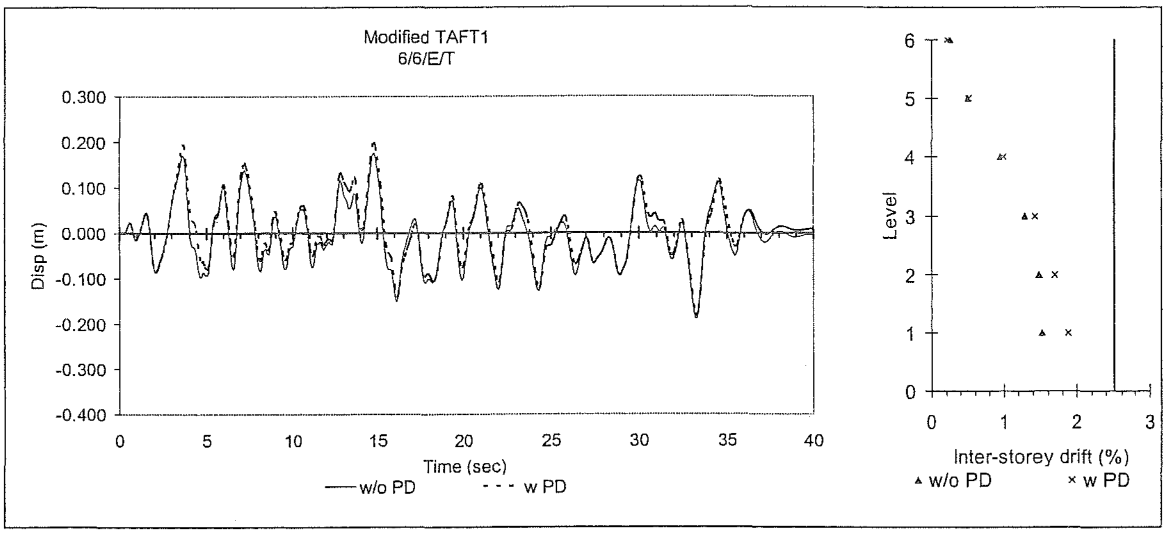




**Figure 6-3 Model 6/6/E/T top floor displacement versus time and maximum inter-storey drifts with and without P-delta for ground acceleration records Modified El Centro EW and Modified TAFT1**

It can be seen that P-delta effects can have an important effect on the maximum inter-storey drifts, especially in the lower levels of the frame. In general, the greater the displacements the greater the P-delta influence. The maximum inter-storey drifts in figure 6.3 for the Modified El Centro EW ground acceleration record show an increment of 10% when P-delta effects are considered. This makes the maximum inter-storey drift at storey 1 to increase from 2.7% to 3.0%. In some cases the influence of P-delta could be the difference between a structure that is near collapse and one that collapses.

It is also important to evaluate P-delta effects when residual displacements are to be predicted. For a complete performance-based design, it is important to have control over the residual displacements (Pampanin et al. 2002). Figure 6.4 shows the residual displacements calculated for structure 6/6/E/T for the four modified records with and without P-delta effects. Note that the scale of the displacements on the horizontal axis is not the same for all graphs.

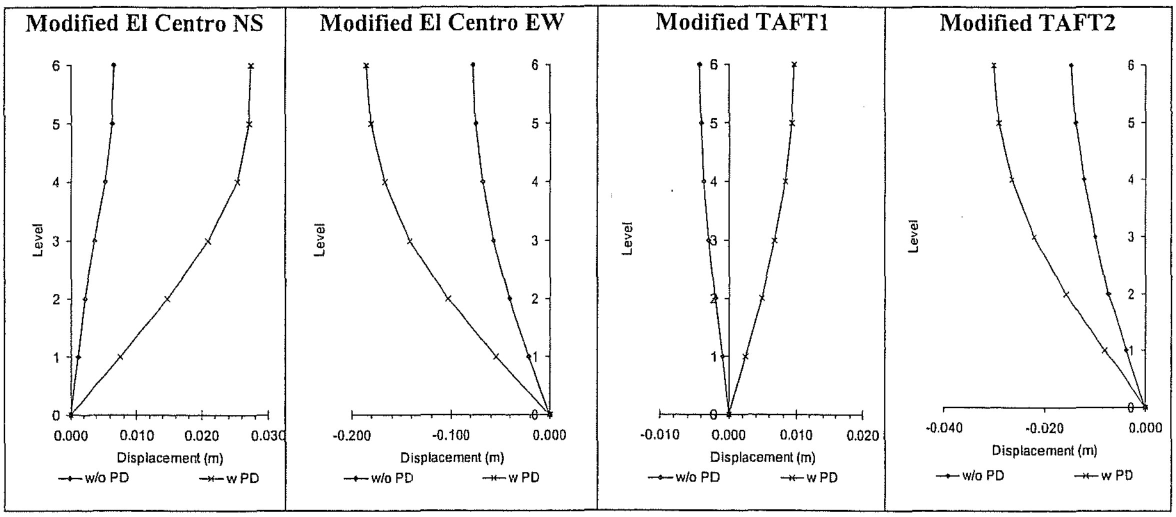



**Figure 6-4 Residual displacements for structure 6/6/E/T**

### *6.3.1 Comparison between frames modelled with a bilinear or a Takeda hysteretic rule*

Many previous analyses were made using a bilinear hysteretic rule. Hence to make use of this previous work it is necessary to have some idea of the significance of using a bilinear hysteresis instead of the more realistic Takeda hysteretic rule. To make an assessment of this, the 6/6/E/T model was reanalysed as a 6/6/E/B model that is using a bilinear hysteretic response using the four modified earthquake records.

Table 6.4 shows that when P-delta effects are omitted, the difference of the maximum inter-storey drifts from the non-linear time history analyses is not significant from structure 6/6/E/T to structure 6/6/E/B. If a ratio of Takeda over bilinear maximum inter-storey drifts is calculated for each floor and then averaged with the other records, the results would be 1.01, 1.02, 1.02, 0.98, 0.90 and 0.89 for storeys 1 through 6 respectively. The maximum inter-storey drifts from levels one through four does not seem to be affected by the choice of hysteretic rule when P-delta effects are not included. Levels 5 and 6 tend to have a more critical inter-storey drift when using the bilinear hysteretic model. The upper storeys are normally not the critical ones and the 11% difference in the inter-storey drift at storey 6 comes from the difference of the averaged maximum inter-storey drift of 0.34% when using the Takeda hysteretic model and the 0.38% for the corresponding values using the bilinear model. For these structures, when P-delta effects are not considered, the more critical response depends on the ground acceleration record used in the analysis rather than the choice of hysteretic rule.

Table 6.5 shows the inter-storey drifts when P-delta effects are considered. A trend can be observed with the more critical inter-storey drifts arising when the bilinear hysteretic model is used. If the same ratio as before is calculated for each level, the average would be 0.92, 0.93, 0.91, 0.87, 0.80 and 0.77 for levels 1 through 6 respectively. A difference of close to a 10% occurs in the response of the 3 lower levels which are the more critical.

**Table 6-4 Maximum inter-storey drifts without considering P-delta effects of model 6/6/E/T and 6/6/E/B**

| *Record* | *Storey* | *Takeda (%)* | *Bilinear (%)* |
|---|---|---|---|
| *Modified El Centro NS* | *1* | 1.28 | 1.45 |
| | *2* | 1.42 | 1.56 |
| | *3* | 1.38 | 1.55 |
| | *4* | 1.06 | 1.22 |
| | *5* | 0.59 | 0.70 |
| | *6* | 0.30 | 0.33 |

| *Record* | *Storey* | *Takeda (%)* | *Bilinear (%)* |
|---|---|---|---|
| *Modified El Centro EW* | *1* | 2.71 | 2.39 |
| | *2* | 2.65 | 2.39 |
| | *3* | 2.26 | 2.06 |
| | *4* | 1.53 | 1.52 |
| | *5* | 0.82 | 0.93 |
| | *6* | 0.41 | 0.44 |

| *Record* | *Storey* | *Takeda (%)* | *Bilinear (%)* |
|---|---|---|---|
| *Modified TAFT1* | *1* | 1.52 | 1.51 |
| | *2* | 1.48 | 1.55 |
| | *3* | 1.28 | 1.33 |
| | *4* | 0.96 | 1.11 |
| | *5* | 0.50 | 0.73 |
| | *6* | 0.26 | 0.37 |

| *Record* | *Storey* | *Takeda (%)* | *Bilinear (%)* |
|---|---|---|---|
| *Modified TAFT2* | *1* | 1.65 | 1.77 |
| | *2* | 1.88 | 1.68 |
| | *3* | 1.93 | 1.68 |
| | *4* | 1.51 | 1.29 |
| | *5* | 0.86 | 0.73 |
| | *6* | 0.38 | 0.38 |

**Table 6-5 Maximum interstorey drifts considering P-delta effects for models 6/6/E/T and 6/6/E/B**

| *Record* | *Storey* | *Takeda (%)* | *Bilinear (%)* |
|---|---|---|---|
| *Modified El Centro NS* | *1* | 1.42 | 1.83 |
| | *2* | 1.50 | 1.92 |
| | *3* | 1.38 | 1.84 |
| | *4* | 1.03 | 1.35 |
| | *5* | 0.56 | 0.71 |
| | *6* | 0.28 | 0.31 |

| *Record* | *Storey* | *Takeda (%)* | *Bilinear (%)* |
|---|---|---|---|
| *Modified El Centro EW* | *1* | 3.06 | 2.97 |
| | *2* | 2.94 | 2.70 |
| | *3* | 2.35 | 2.23 |
| | *4* | 1.54 | 1.56 |
| | *5* | 0.91 | 1.07 |
| | *6* | 0.41 | 0.51 |

| *Record* | *Storey* | *Takeda (%)* | *Bilinear (%)* |
|---|---|---|---|
| *Modified TAFT1* | *1* | 1.88 | 1.73 |
| | *2* | 1.69 | 1.78 |
| | *3* | 1.43 | 1.59 |
| | *4* | 1.01 | 1.26 |
| | *5* | 0.52 | 0.78 |
| | *6* | 0.22 | 0.38 |

| *Record* | *Storey* | *Takeda (%)* | *Bilinear (%)* |
|---|---|---|---|
| *Modified TAFT2* | *1* | 1.83 | 2.28 |
| | *2* | 1.99 | 2.20 |
| | *3* | 1.90 | 2.04 |
| | *4* | 1.41 | 1.49 |
| | *5* | 0.84 | 0.96 |
| | *6* | 0.36 | 0.45 |

*6.3.2 Comparison between frames modelled with different ductilities*

A series of analyses were made to investigate the influence of the design structural ductility factor on the maximum displacements. The 6 storey frames 6/**6**/E/T, 6/**4**/E/T, 6/**2**/E/T and 6/**1**/E/T, all have the same cross-sections, but their individual strengths were found using structural ductility factors of 6, 4, 2 and 1 respectively (as the highlighted numbers indicate). In all cases the columns were elastic except at the bases where a plastic hinge was allowed to develop in each column.

Figure 6.5 and 6.6 show the maximum displacements and inter-storey drifts resulting from non-linear time-history analyses of models 6/**6**/E/T, 6/**4**/E/T, 6/**2**/E/T and 6/**1**/E/T using the modified ground acceleration records with P-delta effects. These models are identified in the graphs by the ductility factor assumed in the design.

It is important to have in mind that these ground acceleration records are modified to fit the design acceleration response spectra and a similar response from one to another was expected. This did hold for three of the records, but not for the Modified El Centro EW record, where the response seems to be more critical. When designing a multi-degree of freedom structure, the choice of ground acceleration records should be carefully studied given that the frequency content of the record can have a significant influence even if the acceleration response spectra for the ground motions in question are similar.

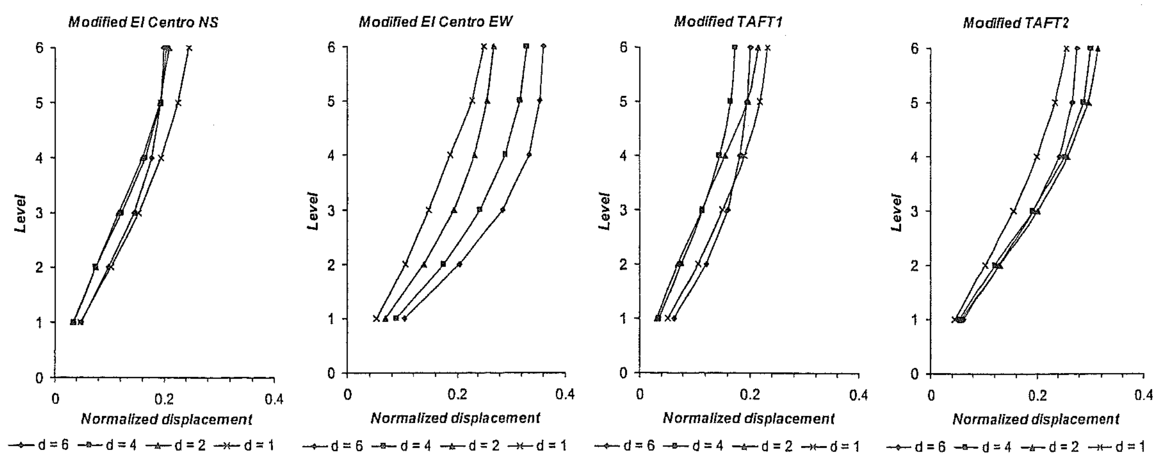



**Figure 6-5 Maximum displacements for six storey models designed to different ductilities**

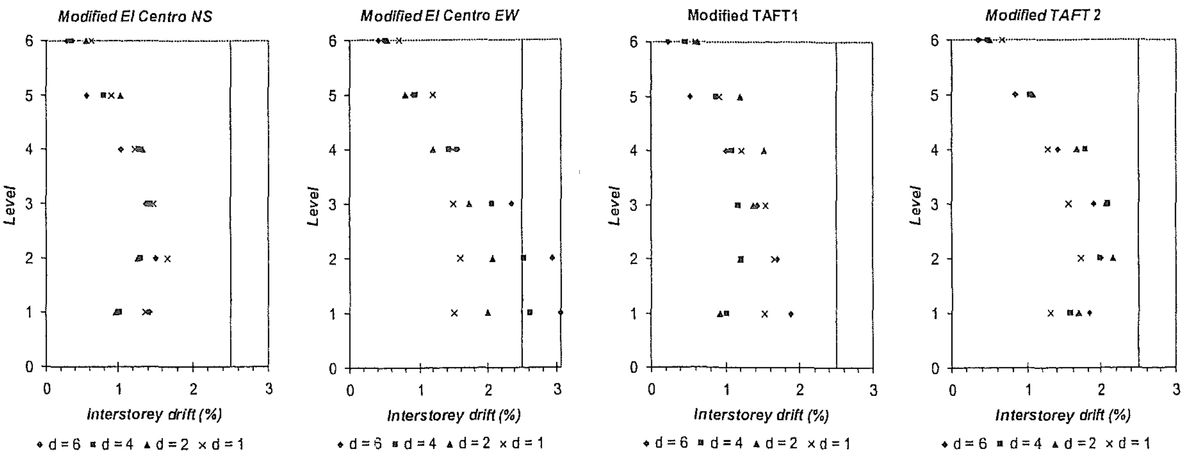



**Figure 6-6 Maximum inter-storey drifts for six storey models designed to different ductilities**

Figure 6.7 shows the averaged inter-storey drifts and maximum displacements sustained with the four different records, both with and without P-delta effects. For these structures, the greatest difference in the maximum inter-storey drifts is at the lower levels. The variation on the maximum storey displacement and the displaced shape is small. The influence to the response of the strength of the beams at the upper levels should not be overlooked. Table 6.6 shows the ratio of the response with P-delta over that without P-delta for the maximum inter-storey drifts and displacements respectively. The lower levels show a difference in the maximum inter-storey drifts from 4% to 8 % for ductilities of 1, 2 and 4 and up to 14% for a ductility of 6. This shows again the influence that P-delta actions have for high ductility levels. The reader should have in mind that even though non-linear time-history analyses are run with and without P-delta effects, all structures were designed to sustain P-delta effects and the strength of its element was chosen accordingly. If one were to compare the response of a structure designed to sustain P-delta effects and one that is not, the difference would be much greater.

**Table 6-6 Ratio between maximum displacements and inter-storey drifts with P-delta effects over maximum displacements and inter-storey drifts without P-delta effects for different ductilities**

| *Inter-storey drifts ratio* | | | | |
|---|---|---|---|---|
| *Storey* | $\mu = 6$ | $\mu = 4$ | $\mu = 2$ | $\mu = 1$ |
| *1* | 1.14 | 1.06 | 1.08 | 1.07 |
| *2* | 1.09 | 1.05 | 1.03 | 1.04 |
| *3* | 1.03 | 1.05 | 1.01 | 1.02 |
| *4* | 0.99 | 0.99 | 0.99 | 1.00 |
| *5* | 1.02 | 0.98 | 0.98 | 0.98 |
| *6* | 0.95 | 1.01 | 0.97 | 0.96 |

| *Maximum displacements ratio* | | | | |
|---|---|---|---|---|
| *Storey* | $\mu = 6$ | $\mu = 4$ | $\mu = 2$ | $\mu = 1$ |
| *1* | 1.14 | 1.06 | 1.08 | 1.07 |
| *2* | 1.12 | 1.05 | 1.06 | 1.06 |
| *3* | 1.09 | 1.06 | 1.03 | 1.03 |
| *4* | 1.06 | 1.05 | 1.02 | 1.02 |
| *5* | 1.05 | 1.04 | 1.01 | 1.02 |
| *6* | 1.04 | 1.04 | 1.01 | 1.02 |

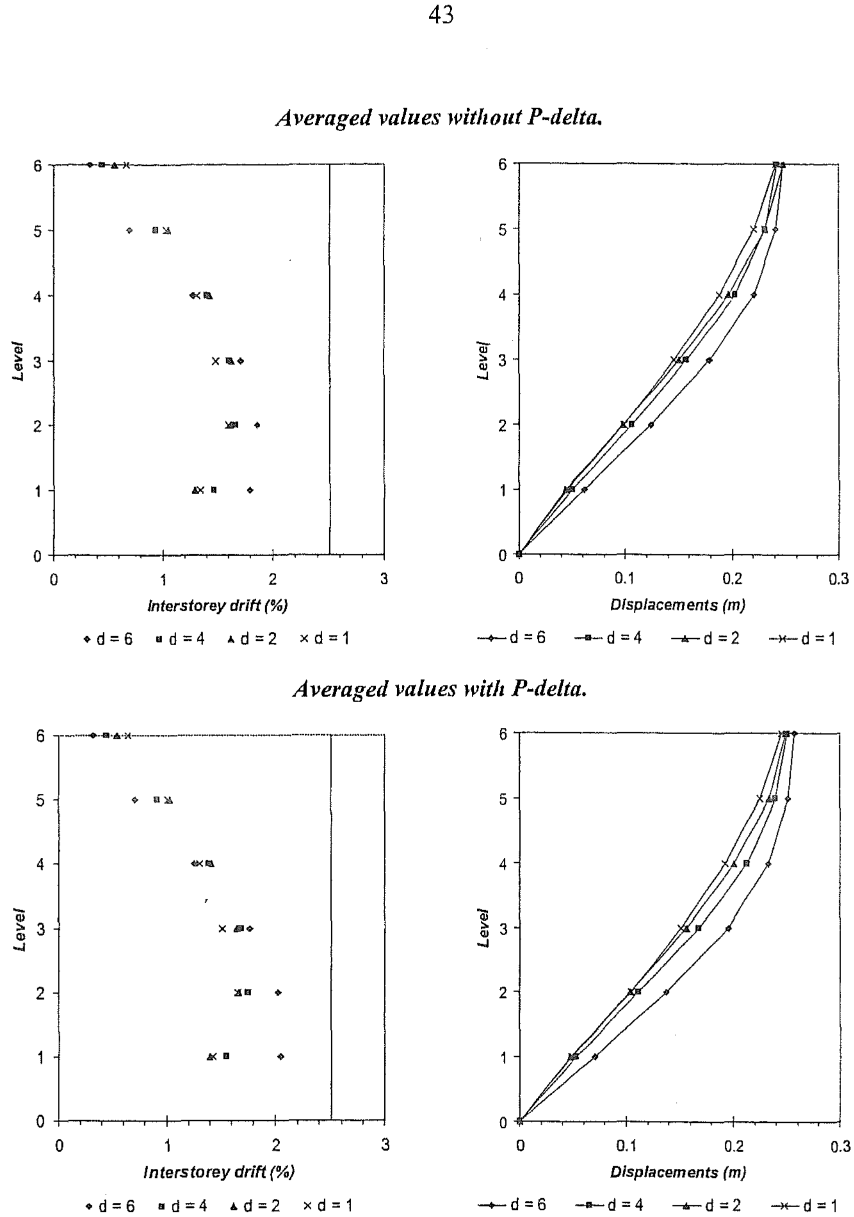



**Figure 6-7 Averaged inter-storey drifts and maximum displacements for models 6/6/E/T, 6/4/E/T, 6/2/E/T and 6/1/E/T with and without P-delta**

When evaluating P-delta effects using the commentary to the Loadings Standard (Standards New Zealand. 1992) it is suggested that P-delta induced forces should be calculated using the maximum displacements. To verify if there would be a difference between assessing P-delta forces using the maximum displacements or the maximum inter-storey drifts, a comparison was made between the maximum inter-storey drifts and the inter-storey drifts resulting from the envelope deflected shapes for the non-linear time-history analyses using the four modified ground acceleration records including P-delta effects. Table 6.7 shows the average value for this comparison using models 6/6/E/T, 6/4/E/T, 6/2/E/T and 6/1/E/T for the four modified ground acceleration records. The averaged inter-storey drifts coming from the envelope displacements are very similar to the averaged maximum inter-storey drifts at the lower levels where P-delta effects are more critical and hence the difference between assessing P-delta effects with maximum displacements or maximum inter-storey drifts would be negligible.

**Table 6-7 Comparison between maximum displacement and maximum inter-storey drifts**

| *Storey* | *Disp. / Drift* | | | |
|---|---|---|---|---|
| | $\mu = 6$ | $\mu = 4$ | $\mu = 2$ | $\mu = 1$ |
| *1* | 1.00 | 1.00 | 1.00 | 1.00 |
| *2* | 0.99 | 0.97 | 0.99 | 0.99 |
| *3* | 0.94 | 0.98 | 0.93 | 0.91 |
| *4* | 0.87 | 0.95 | 0.93 | 0.93 |
| *5* | 0.78 | 0.87 | 0.93 | 0.95 |
| *6* | 0.56 | 0.70 | 0.84 | 0.91 |

### *6.3.3 Axial load variation of interior columns*

Because only the rotations of interior columns will be further studied, some analyses were made to verify the axial load variation in them. For information on the axial load variation of exterior columns refer to previous works (Kelly 1974; Lindup 1975; Moss and Carr 1980; Row 1973; Tompkins 1980).

Given the regular configuration of the frame and the low axial gravity load acting on the columns, the axial load variation when a non-linear time-history analysis is run was expected to be small.

Table 6.8 shows the design axial loads and the maximum and minimum axial loads at an internal column from the non-linear time-history analyses using the four modified ground acceleration records. These results are for model 6/6/3101/T. It can be seen that the interior columns remain in compression at all times as expected. The most critical axial load will be the one with the lowest compression.

**Table 6-8 Axial load variation in non-linear time-history analyses**

| *Level* | *Design load* | *Ground acceleration record (All loads are in kN)* | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | *1* | | *2* | | *3* | | *4* | |
| | | *Max* | *Min* | *Max* | *Min* | *Max* | *Min* | *Max* | *Min* |
| *1* | -512.5 | -485.2 | -572.2 | -501.0 | -589.4 | -511.0 | -595.6 | -515.3 | -578.3 |
| *2* | -425.5 | -424.9 | -471.8 | -407.9 | -478.6 | -427.6 | -482.1 | -420.7 | -476.0 |
| *3* | -344.0 | -336.1 | -373.6 | -315.1 | -380.0 | -341.2 | -383.9 | -331.4 | -381.0 |
| *4* | -259.7 | -239.2 | -274.6 | -224.4 | -286.0 | -250.4 | -288.0 | -239.9 | -285.1 |
| *5* | -177.0 | -144.3 | -182.3 | -131.8 | -192.4 | -162.0 | -190.0 | -148.9 | -190.1 |

**Negative sign indicates compression.*

Table 6.9 shows the relation between the design axial load and the maximum axial load (which represents the lowest compression) for each record, the design reinforcing steel content for each level ($\rho_{design}$) and the steel content needed if the columns would have been designed using the most critical axial load from the non-linear time-history analyses. It is clear that the variation of axial load had little effect on the column.

**Table 6-9 Reinforcement ratio variation for most critical axial load in interior columns for non-linear time-history analyses**

| | *Design / Max* | | | | *Steel content* | |
|---|---|---|---|---|---|---|
| | *Ground acceleration record* | | | | $\rho_{design}$ | $\rho_{new}$ |
| *Level* | 1 | 2 | 3 | 4 | | |
| *1* | **1.06** | 1.02 | 1.00 | 0.99 | 0.0170 | 0.0173 |
| *2* | 1.00 | **1.04** | 1.00 | 1.01 | 0.0207 | 0.0210 |
| *3* | 1.02 | **1.09** | 1.01 | 1.04 | 0.0180 | 0.0183 |
| *4* | 1.09 | **1.16** | 1.04 | 1.08 | 0.0205 | 0.0207 |
| *5* | 1.23 | **1.34** | 1.09 | 1.19 | 0.0183 | 0.0185 |

### *6.3.4 Comparison between frames with different column strengths*

In this section a summary of the results from the non-linear time-history analyses of models 6/6/E/T, 6/6/3101/T and 6/6/1170/T using the eight ground acceleration records described in section 5.13.2 is presented. The main characteristics of these models are listed below. For more details on the properties of the structures see section 5.1 and appendix A.

1. Structure 6/6/E/T
    - Designed to a ductility of 6
    - All beams comply with minimum steel requirements
    - Beams at levels 3, 4, 5 and 6 have the same strength
    - Columns are elastic except at the base
    - A Takeda hysteretic model is used for all elements

2. Structure 6/6/3101/T
    - Designed to a ductility of 6
    - All beams comply with minimum steel requirements
    - Beams at levels 3, 4, 5 and 6 have the same strength
    - Columns are designed to NZS 3101:1995 (Standards New Zealand. 1995)
    - A Takeda hysteretic model is used for all elements

3. Structure 6/6/1170/T

- Designed to a ductility of 6
- All beams comply with minimum steel requirements
- Beams at levels 3, 4, 5 and 6 have the same strength
- Columns are designed to NZS 1170.5 draft for limited protection against plastic hinges
- A Takeda hysteretic model is used for all elements

Appendix B, part one, shows the response of structures 6/6/E/T, 6/6/3101/T and 6/6/1170/T from the non-linear time-history analyses using each of the selected ground acceleration records. The results given in this appendix for each structure for each of the ground acceleration records are:

1. The maximum base shear with and without P-delta effects
2. A graph and the values of the maximum inter-storey drifts with and without P-delta effects
3. A graph showing the maximum displacements with and without P-delta effects
4. The maximum rotation of plastic hinge on the left side of the beams in the central span
5. The maximum rotation of the plastic hinge on one of the interior columns for levels one through six.

The rotations column rotations are reported at ground level (G. L.) and below and above the beam face at all levels. P-delta effects are always considered to act in the calculations for the maximum rotations.

*6.3.4.1 Base shear*

Table 6.10 shows the average peak base shear and the standard deviation for the 8 records without and with P-delta effects respectively. It shows as well the Equivalent static and combined modal base shear values and the sum of the forces added in each analysis to account for P-delta effects.

**Table 6-10 Base shear summary for structures 6/6/E/T, 6/6/3101/T and 6/6/1170/T**

| | Eq. Static (kN) | Modal (kN) |
|---|---|---|
| ***Base shear*** | 732.2 | 631.0 |
| ***P-Δ induced**** | 254.8 | 253.9 |
| $\Sigma=$ | 987.0 | 884.9 |

| | 6/6/E/T (kN) | | 6/6/3101/T (kN) | | 6/6/1170/T (kN) | |
|---|---|---|---|---|---|---|
| | **Average** | **Std. Dev.** | **Average** | **Std. Dev.** | **Average** | **Std. Dev.** |
| ***Without PD*** | 1418.3 | 88.9 | 1411.1 | 86.5 | 1326.4 | 64.0 |
| ***With PD*** | 1584.9 | 199.7 | 1579.6 | 193.7 | 1443.9 | 120.8 |

**P-delta forces are already multiplied by the factor β.*

An increase of around two fold for the modal base shear is observed when compared to the averaged results from the non-linear time-history analyses when P-delta effects are considered. The increase in base shear above the design level comes from the difference in shear distribution over the length of the building observed in the elastic analyses (equivalent static and modal) and the strength increases in the beam due to the strength reduction factor, the minimum longitudinal reinforcement requirements and strain hardening of the reinforcement. This observed increase in the base shear should not represent a problem if the columns have enough shear strength to sustain it. Appendix C shows the effect that damping forces have on the base shear. It can be seen that damping forces have a considerable magnitude and the peak base shear values could be significantly affected by them. Immediately after the time when maximum base shears are present, the structure reverses its direction which means that at that point its velocity is close to zero so that the damping forces are minimal.

For structures 6/6/E/T, 6/6/3101/T and 6/6/1170/T the increase on the average peak base shear when P-delta effects are considered is of 11.7%, 11.9% and 8.8 % respectively compared to that when P-delta is not considered. There is a small difference in the average peak base shear between structures 6/6/E/T and 6/6/3101/T which represents the little inelastic behaviour shown in columns of model 6/6/3101/T. As expected, there is a reduction when comparing the averaged maximum base shear from structure 6/6/3101/T to that from structure 6/6/1170/T. This reduction can be related to the energy dissipation coming from the plastic hinging in columns above the base. When the average maximum base shear for the three structures using the four modified records are compared with the corresponding values for the four natural records it is found that the former are 7%, 11% and 8% greater than the latter for each of the structures respectively.

### *6.3.4.2 Inter-storey drifts*

Table 6.11 shows the average maximum inter-storey drifts for the eight ground acceleration records with P-delta effects for structures 6/6/E/T, 6/6/3101/T and 6/6/1170/T and its standard deviation. The average at all levels is below the accepted 2.5% being the most critical value the 2.02% inter-storey drift of structure 6/6/1170/T. The most critical averaged maximum inter-storey drift of of structures 6/6/E/T and 6/6/3101/T (1.68% and 1.81% respectively), do not exceed the maximum inter-storey drift limit of 1.82% for the elastic analyses for these structures. The averaged maximum inter-storey drift of structure 6/6/1170/T exceeds this limit by 11%.

**Table 6-11 Average maximum inter-storey drifts and standard deviations considering P-delta effects using eight ground acceleration records for structures 6/6/E/T, 6/6/3101/T and 6/6/1170/T**

| | | *6/6/E/T* | | *6/6/3101/T* | | *6/6/1170/T* | |
|---|---|---|---|---|---|---|---|
| | | *Avg.* | *Std dev.* | *Avg.* | *Std. dev.* | *Avg.* | *Std. dev.* |
| | *Level* | (%) | (%) | (%) | (%) | (%) | (%) |
| *With P-delta* | *1* | 1.57 | 0.60 | 1.75 | 0.73 | 1.34 | 0.77 |
| | *2* | 1.68 | 0.49 | 1.81 | 0.59 | 1.83 | 0.71 |
| | *3* | 1.54 | 0.38 | 1.60 | 0.43 | 2.02 | 0.56 |
| | *4* | 1.18 | 0.27 | 1.15 | 0.24 | 1.68 | 0.40 |
| | *5* | 0.71 | 0.19 | 0.68 | 0.18 | 0.75 | 0.19 |
| | *6* | 0.36 | 0.08 | 0.37 | 0.10 | 0.40 | 0.10 |

Table 6.12 shows the ratio between the averaged maximum inter-storey drift with P-delta effects over the corresponding value without P-delta effects. An increase of 12% for structures 6/6/E/T and 6/6/3101/T and 15% for structure 6/6/1170/T is observed when P-delta effects are considered.

Figure 6.8 shows the maximum inter-storey drifts from the non-linear time-history analyses using the eight records for the three structures, considering P-delta effects. The average of these values is also illustrated. Structures 6/6/3101/T and 6/6/1170/T only exceeded the 2.5% maximum inter-storey drift limit when using the modified El Centro EW ground acceleration record. The high ductilities shown in the lower storeys, represented by the high inter-storey drifts, create a base isolation effect at the upper storeys. Appendix D shows the displacement response spectra for the ground acceleration records in an attempt to explain the scatter of the inter-storey drifts and displacements.

**Table 6-12 Comparison between the maximum average inter-storey drifts with P-delta and without P-delta for structures 6/6/E/T, 6/6/3101/T and 6/6/1170/T**

| *Averaged maximum Inter-storey drifts With P-delta / Without P-delta* | | | |
|---|---|---|---|
| *Level* | *6/6/E/T* | *6/6/3101/T* | *6/6/1170/T* |
| *1* | 1.12 | 1.12 | 1.10 |
| *2* | 1.08 | 1.09 | 1.13 |
| *3* | 1.03 | 1.03 | 1.15 |
| *4* | 1.00 | 0.98 | 1.08 |
| *5* | 0.98 | 0.97 | 0.92 |
| *6* | 0.94 | 0.93 | 0.93 |

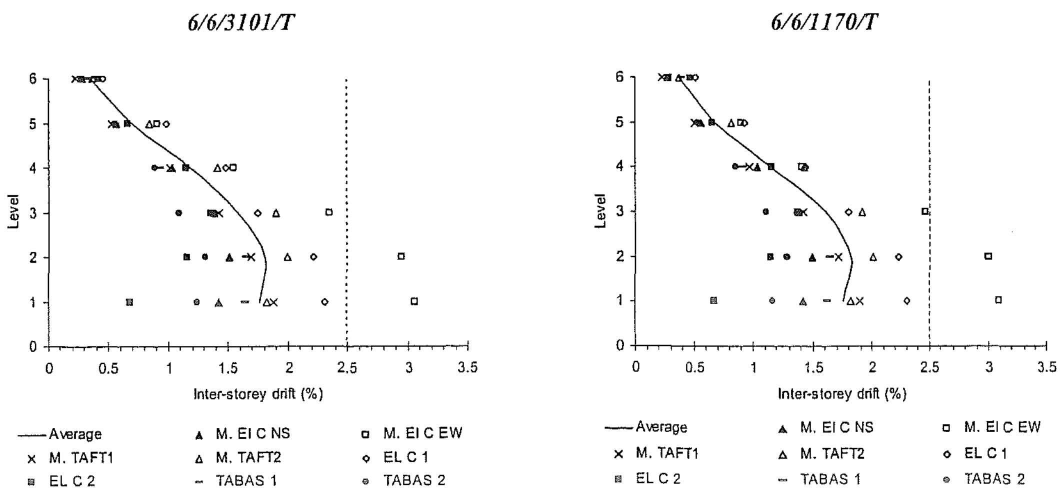



**Figure 6-8 Maximum inter-storey drifts and its average for the non-linear time-history analyses of structures 6/6/3101/T and 6/6/1170/T with P-delta effects**

Table 6.13 shows a comparison of the averaged maximum inter-storey drifts coming from the four non-linear time-history analyses using the modified ground acceleration records and the corresponding values obtained using the natural ground acceleration records. Structures 6/6/3101/T and 6/6/1170/T show an increase of up to 42% and 62% respectively when using the modified records.

**Table 6-13 Averaged maximum inter-storey drifts using the modified ground acceleration records compared to those using the natural ground acceleration records for structures 6/6/E/T, 6/6/3101/T and 6/6/1170/T**

| *Level* | *6/6/E/T* | *6/6/3101/T* | *6/6/1170/T* |
|---|---|---|---|
| | *Modified / Natural* | *Modified / Natural* | *Modified / Natural* |
| *1* | 1.40 | 1.42 | 1.62 |
| *2* | 1.28 | 1.30 | 1.43 |
| *3* | 1.26 | 1.26 | 1.26 |
| *4* | 1.13 | 1.11 | 1.09 |
| *5* | 1.03 | 1.05 | 0.84 |
| *6* | 0.87 | 0.81 | 0.89 |

### *6.3.4.3 Maximum displacements*

Table 6.14 shows the averaged envelope displacements and standard deviations of structures 6/6/E/T, 6/6/3101/T and 6/6/1170/T from the non-linear time-history analyses using the eight ground acceleration records considering P-delta effects. The results from structures 6/6/E/T and 6/6/3101/T show no significant variation.

**Table 6-14 Averaged maximum displacements of structures 6/6/E/T, 6/6/3101/T and 6/6/1170/T with P-delta effects**

| **Level** | *6/6/E/T* | | *6/6/3101/T* | | *6/6/1170/T* | |
|---|---|---|---|---|---|---|
| | **Average (m)** | **Std Deviation (m)** | **Average (m)** | **Std Deviation (m)** | **Average (m)** | **Std Deviation (m)** |
| 1 | 0.060 | 0.024 | 0.059 | 0.025 | 0.046 | 0.026 |
| 2 | 0.121 | 0.044 | 0.121 | 0.045 | 0.106 | 0.051 |
| 3 | 0.172 | 0.056 | 0.173 | 0.058 | 0.171 | 0.068 |
| 4 | 0.206 | 0.063 | 0.207 | 0.063 | 0.226 | 0.076 |
| 5 | 0.225 | 0.066 | 0.225 | 0.066 | 0.246 | 0.078 |
| 6 | 0.232 | 0.067 | 0.232 | 0.068 | 0.253 | 0.079 |

Figure 6.9 shows a comparison between the averaged envelope of displacements found in the non-linear time-history analyses considering P-delta effects for structures 6/6/E/T, 6/6/3101/T and 6/6/1170/T and the inelastic displacements considering P-delta effects for the combined modal analysis. There is a maximum increase of 28% on the first level when the averaged maximum displacements are compared to the modal with P-delta displacements of structures 6/6/E/T and 6/6/3101/T. For structure 6/6/1170/T a maximum increase of up to 10% on the 5$^{th}$ level is observed.

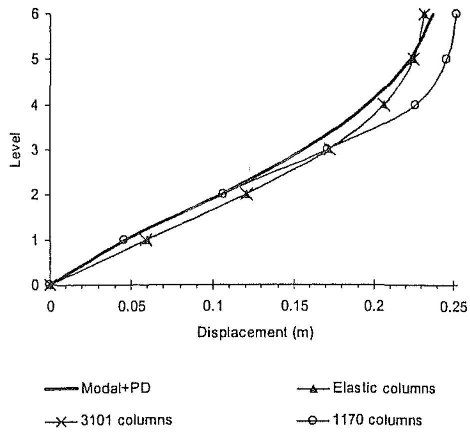



**Figure 6-9 Non-linear time-history analyses averaged maximum displacements of structures 6/6/E/T, 6/6/3101/T and 6/6/1170/T considering P-delta effects compared to modal displacements plus P-delta effects**

Figure 6.10 illustrates the scatter of maximum displacements for the eight ground acceleration records and the modal plus P-delta actions deformed shape. Again the amplification of the response when using the modified ground acceleration records can be observed when compared to the response when using the natural ground acceleration records.

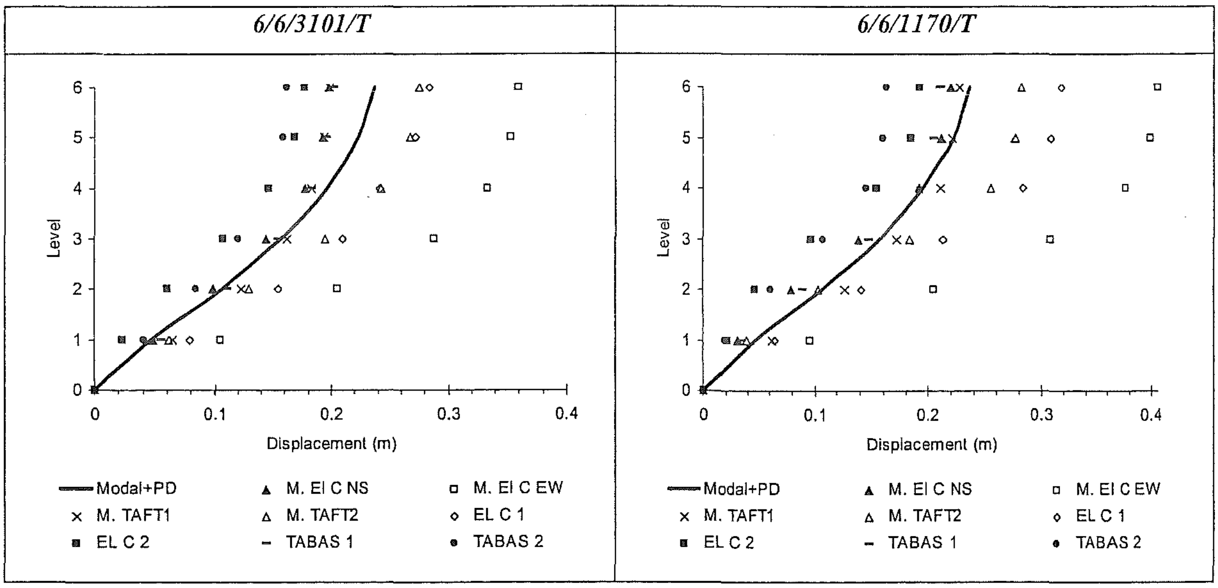



**Figure 6-10 Maximum displacements of structures 6/6/3101/T and 6/6/1170/T for eight ground acceleration records**

### *6.3.4.4 Beam rotations*

The maximum curvature of the elements for the non-linear time-history analyses was obtained directly from the structural analysis program (Carr 2003). Hence, the maximum rotations were calculated by multiplying the maximum curvature of the element by the length of its corresponding plastic hinge.

In this section, the maximum beam rotations from the non-linear time-history analyses are compared with the design beam rotations implied in the equivalent static and modal analyses. To obtain the rotations from the equivalent static and modal analyses, the following procedure was followed.

To find the total rotation demand of a plastic hinge in a beam of a frame when subject to a static lateral load, the beam rotation induced from both the elastic and the inelastic deformations need to be individually assessed and then added together. As shown in figure 6.11.a, when a frame structure is subject to a lateral force, the rotations while the structure remains on the elastic range are taken by the beams and the columns. Once inelastic behaviour is reached, the rotations concentrate on the plastic hinge zones (figure 6.11.b).

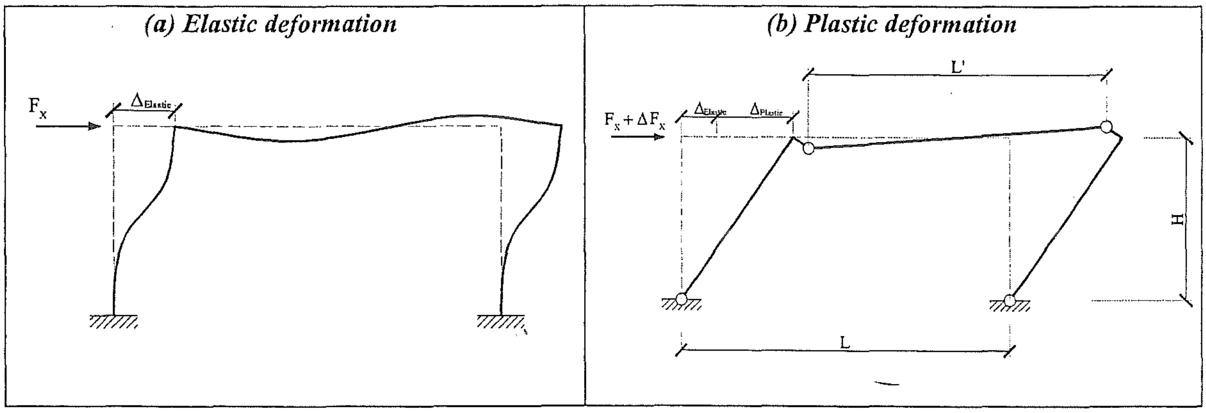



**Figure 6-11 Elastic and inelastic deformed shape of a frame**

The plastic rotation in a beam plastic hinge zone can be calculated by considering the deformed shape assuming that all angles are small. Thus, with reference to Figure 6.11.b, the plastic rotation, $\theta_{plastic}$, is given by:

$$\theta_{plastic} = \frac{\Delta_{plastic}}{H} x \frac{L}{L'}$$

where $\Delta_{plastic}$ is the plastic lateral displacement, $H$ is the storey height, $L$ is the distance between column centrelines and $L'$ is the distance between column faces.

To assess the plastic rotation from an equivalent static or a modal analyses, the plastic inter-storey drift ($\Delta_{plastic} / H$) is obtained by multiplying the maximum inter-storey drift by $\mu - 1$, where $\mu$ is the design ductility factor.

The allowable inter-storey drift limits applying to an equivalent static or a modal analysis differ from the limit for a non-linear time-history analysis. The reason for this is that elastic based methods of analysis have been found to underestimate inter-storey drifts. In order to compare the design rotations from an elastic analysis to those from a non-linear analysis, the elastic rotations should be scaled by a factor $f$, of:

$$f = {ID_{Plastic}}/{ID_{Elastic}}$$

where $ID_{plastic}$ is the maximum allowable inter-storey drift for the non-linear analysis and $ID_{Elastic}$ the maximum allowable inter-storey drift for an elastic analysis.

Table 6.15 shows the total beam rotation demand from the equivalent static and modal analyses multiplied by the factor f described earlier of 1.37 *(2.5/1.82 = 1.37)*. The average of the maximum beam rotations for the non-linear time-history analyses using the eight ground acceleration records for structures 6/6/E/T, 6/6/3101/T and 6/6/1170/T and its standard deviations are also shown. In all cases when calculating the rotations, P-delta effects were considered. These same average maximum beam rotations are shown in figure 6.12.

**Table 6-15 Equivalent static and modal analysis beam rotation demand and average of maximum beam rotations and standard deviations from the non-linear time-history analyses of structures 6/6/E/T, 6/6/3101/T and 6/6/1170/T**

| *Level* | *Eq. static rotations (rad)* | *Modal rotations (rad)* | *6/6/E/T* | | *6/6/3101/T* | | *6/6/1170/T* | |
|---|---|---|---|---|---|---|---|---|
| | | | *Average rot. (rad)* | *Std. Dev. (rad)* | *Average rot. (rad)* | *Std. Dev. (rad)* | *Average rot. (rad)* | *Std. Dev. (rad)* |
| 1 | 0.0185 | 0.0184 | 0.0165 | 0.0072 | 0.0164 | 0.0072 | 0.0116 | 0.0074 |
| 2 | 0.0243 | 0.0232 | 0.0157 | 0.0053 | 0.0159 | 0.0056 | 0.0160 | 0.0055 |
| 3 | 0.0219 | 0.0200 | 0.0128 | 0.0036 | 0.0125 | 0.0031 | 0.0155 | 0.0035 |
| 4 | 0.0177 | 0.0155 | 0.0076 | 0.0024 | 0.0075 | 0.0023 | 0.0046 | 0.0021 |
| 5 | 0.0128 | 0.0109 | 0.0031 | 0.0015 | 0.0030 | 0.0014 | 0.0005 | 0.0000 |
| 6 | 0.0082 | 0.0064 | 0.0004 | 0.0001 | 0.0003 | 0.0000 | 0.0001 | 0.0000 |

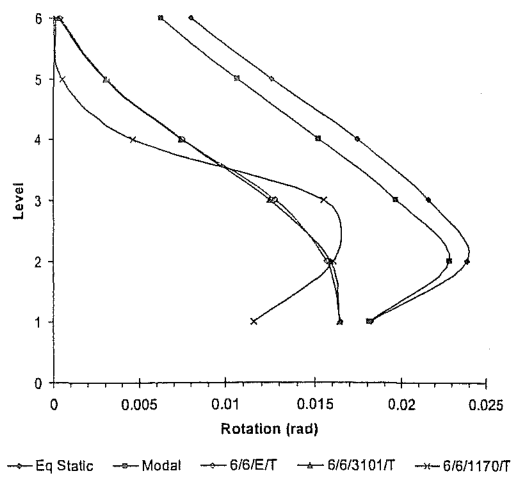



**Figure 6-12 Equivalent static and modal analysis beam rotation demand and average of maximum beam rotations from the non-linear time-history analyses of structures 6/6/E/T, 6/6/3101/T and 6/6/1170/T**

From this it is clear that the average beam rotation demand for the three structures is below the beam rotation demand from both the equivalent and modal analyses.

Again, the difference between averaged maximum beam rotations of structures 6/6/E/T and 6/6/3101/T show no significant variation. The average maximum beam rotations of structure 6/6/3101/T and 6/6/1170/T show that for the first storey the beam rotation demand is greater by up to a 42% for model 6/6/3101/T. At the second storey the rotations for both models are very similar and at the third storey structure 6/6/1170/T shows a 19% greater beam rotation demand. At the upper half of the structure, the averaged maximum beam rotations of model 6/6/3101/T are much greater than for model 6/6/1170/T. The beam rotations at level six are small given that yielding of columns is allowed on the top floor.

The maximum beam rotation for both structures 6/6/3101/T and 6/6/1170/T were obtained when using the modified record El Centro EW. If the beam rotations from this record are compared to those from the equivalent static analysis, the two lower levels exceed the design beam rotations by 39% and 12% for structure 6/3/3101/T and 27% and 8% for structure 6/6/1170/T. Above level 3 beam rotations are below those from the equivalent static and modal analyses for all ground acceleration records.

Figure 6.13 shows the maximum beam rotations of structures 6/6/3101/T and 6/6/1170/T for the non-linear time-history analyses using the eight ground acceleration records. This illustrates how scattered the rotations are, especially in the bottom half of both structures. Table 6.16 shows the averaged maximum beam section ductility for the eight non-linear time-history analyses of structures 6/6/E/T, 6/6/3101/T and 6/6/1170/T. Maximum section ductility values of close to 20 are observed for the three structures.

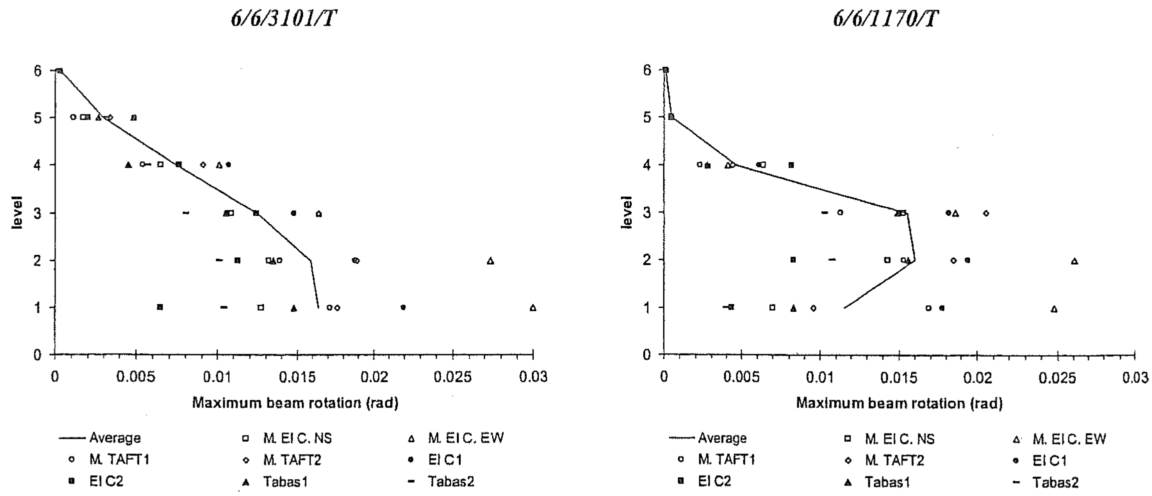



Figure 6-13 Maximum beam rotations of structures 6/6/3101/T and 6/6/1170/T from non-linear time-history analyses

### *6.3.4.5 Interior column rotations*

The maximum column rotations from the non-linear time-history analyses were obtained by multiplying the maximum curvature value supplied by the structural analysis computer program by the length of its corresponding plastic hinge. The yield rotation values were calculated with the help of a

**Table 6-16 Average of maximum beam section rotation ductilities**

| Level | Eq static | Modal | 6/6/E/T | 6/6/3101/T | 6/6/1170/T |
|---|---|---|---|---|---|
| | $S_\mu$ | $S_\mu$ | $S_\mu$ | $S_\mu$ | $S_\mu$ |
| 1 | 22.3 | 22.1 | 19.8 | 19.7 | 13.9 |
| 2 | 30.0 | 28.7 | 19.3 | 19.6 | 19.8 |
| 3 | 32.9 | 30.0 | 19.2 | 18.8 | 23.3 |
| 4 | 26.5 | 23.2 | 11.3 | 11.2 | 7.0 |
| 5 | 19.2 | 16.3 | 4.6 | 4.5 | 0.8 |
| 6 | 12.2 | 9.5 | 0.5 | 0.4 | 0.2 |

spread sheet. Table 6.17 shows the yield rotation, $\theta_y$, the averaged maximum rotation for the eight non-linear time-history analyses, its standard deviation and the averaged maximum curvature section ductility, $S_\mu$, for an interior column of structures 6/6/3101/T and 6/6/1170/T respectively. For structure 6/6/3101/T, the restriction for the formation of plastic hinges in columns except at the base and the top floor was closely met by the averaged maximum rotation values which show a section rotation ductility lower than 2. For structure 6/6/1170/T that allows for inelastic behaviour of columns, the greatest column section ductility was at level 4 rather than at the base of the column. As mentioned before, the maximum rotations from the non-linear time-history analyses were obtained considering P-delta effects.

**Table 6-17 Interior columns design yield rotation, averaged maximum rotation and standard deviation for eight ground acceleration records for structures 6/6/3101/T and 6/6/1170/T**

| *level* | *6/6/3101/T* | | | | *6/6/1170/T* | | | |
|---|---|---|---|---|---|---|---|---|
| | *$\theta_y$ (rad)* | *Avg. rot (rad)* | *Std. dev. (rad)* | *$S_\mu$* | *$\theta_y$ (rad)* | *Avg. rot (rad)* | *Std. dev. (rad)* | *$S_\mu$* |
| ***G. L.*** | 0.0012 | 0.0153 | 0.0071 | 12.27 | 0.0012 | 0.0116 | 0.0071 | 9.34 |
| ***1 Below*** | 0.0011 | 0.0011 | 0.0002 | 0.92 | 0.0012 | 0.0013 | 0.0002 | 1.05 |
| ***1 Above*** | 0.0011 | 0.0014 | 0.0002 | 1.25 | 0.0012 | 0.0070 | 0.0002 | 5.77 |
| ***2 Below*** | 0.0011 | 0.0015 | 0.0003 | 1.33 | 0.0012 | 0.0038 | 0.0003 | 3.15 |
| ***2 Above*** | 0.0011 | 0.0012 | 0.0002 | 1.03 | 0.0012 | 0.0047 | 0.0002 | 3.88 |
| ***3 Below*** | 0.0011 | 0.0021 | 0.0018 | 1.95 | 0.0012 | 0.0059 | 0.0018 | 5.02 |
| ***3 Above*** | 0.0011 | 0.0010 | 0.0002 | 0.87 | 0.0012 | 0.0033 | 0.0002 | 2.79 |
| ***4 Below*** | 0.0011 | 0.0015 | 0.0002 | 1.37 | 0.0011 | 0.0122 | 0.0002 | 10.89 |
| ***4 Above*** | 0.0011 | 0.0007 | 0.0001 | 0.66 | 0.0011 | 0.0034 | 0.0001 | 3.04 |
| ***5 Below*** | 0.0011 | 0.0013 | 0.0002 | 1.21 | 0.0011 | 0.0070 | 0.0002 | 6.51 |
| ***5 Above*** | 0.0011 | 0.0006 | 0.0001 | 0.51 | 0.0011 | 0.0031 | 0.0001 | 2.83 |
| ***6 Below*** | 0.0009 | 0.0025 | 0.0012 | 2.76 | 0.0009 | 0.0049 | 0.0012 | 5.27 |

Figure 6.14 shows the column yield rotation and the maximum rotation from the eight non-linear time-history analyses of structure 6/6/3101/T. The maximum column rotations below the beam at

level three show a great dispersion that affects considerably the average maximum rotation and the standard deviation at this level. The maximum section rotation ductility for column sections below the beam is of 5.9 at level 3 when using the modified ground acceleration record El Centro EW. The maximum section rotation ductility for column sections above the beam is of 1.6 at level 1 when using the ground acceleration record El Centro 2.

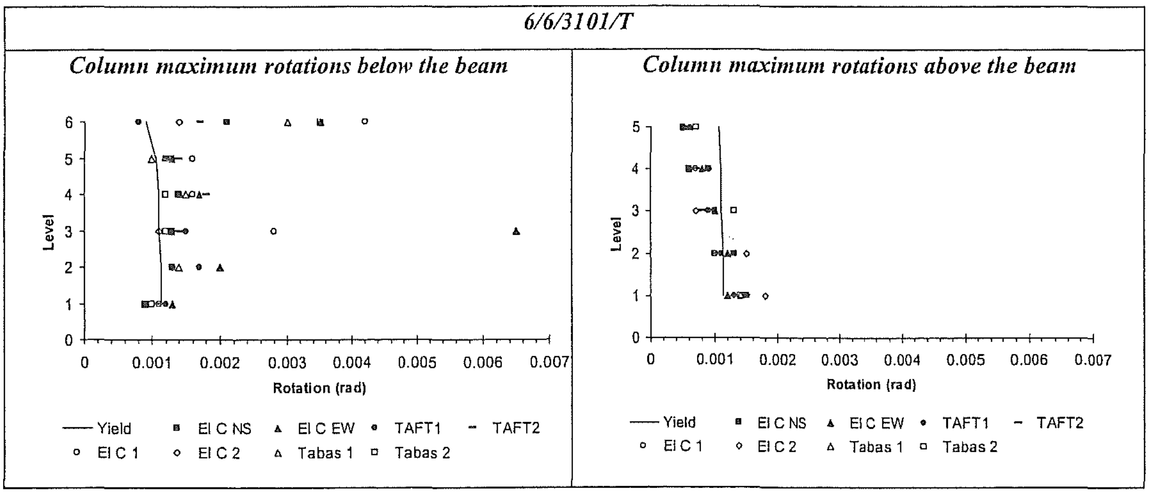



**Figure 6-14 Maximum column rotations for non-linear time-history analyses to structure 6/6/3101/T**

Figure 6.15 shows the column yield rotation and the maximum rotation from the eight non-linear time-history analyses of structure 6/6/1170/T. The maximum section rotation ductility for column sections below the beam is of 16 at level 4 when using the ground acceleration record TAFT 2. The maximum section rotation ductility for column sections above the beam is of 8.0 at level 1 when using the modified ground acceleration records El Centro EW and TAFT 2.

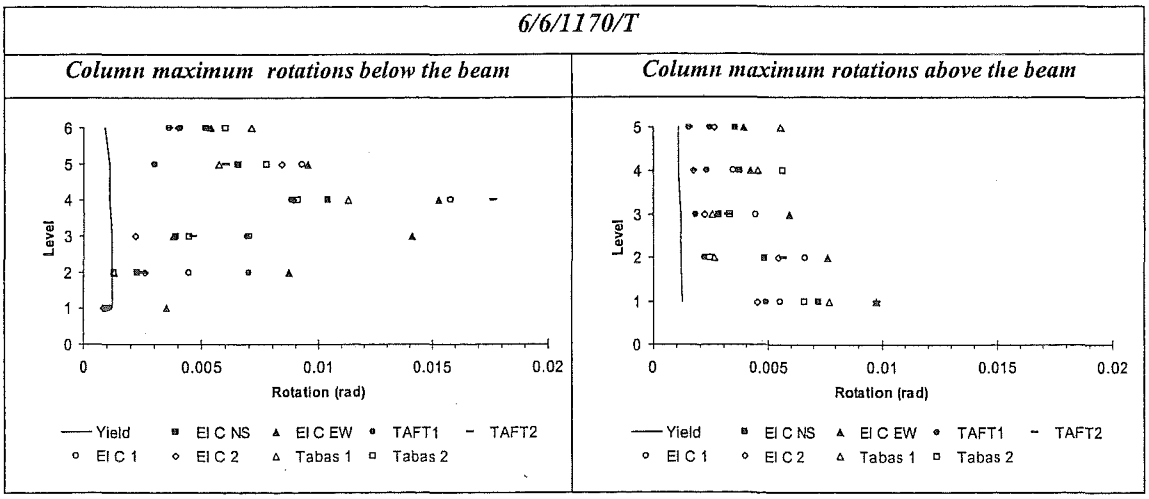



**Figure 6-15 Maximum column rotations for non-linear time-history analyses to structure 6/6/1170/T**

Figure 6.16 compares the average maximum column rotation values below and above the beams, for the non-linear time-history analyses of structures 6/6/3101/T and 6/6/1170/T. For both structures, the column rotations below the beams show a considerable increase at levels 3 to 5 from those above the beams.

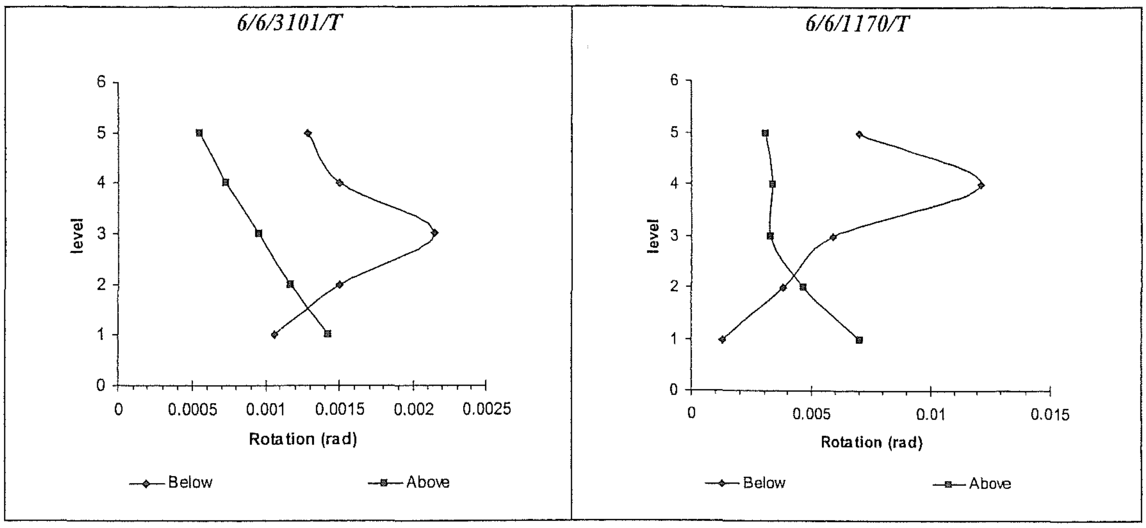



**Figure 6-16 Comparison between average maximum rotations below and above the beam for structures 6/6/3101/T and 6/6/1170/T**

### *6.3.5 Earthquake return period of 2,500 years*

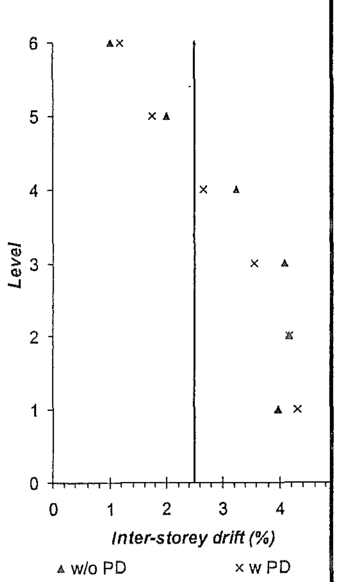
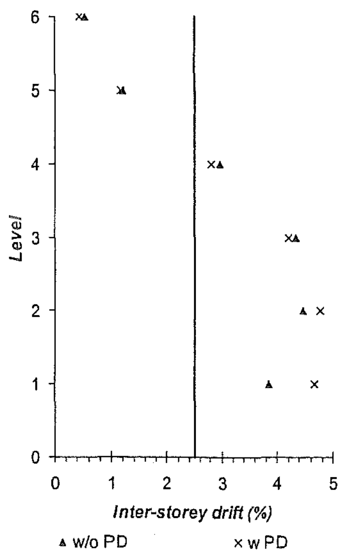
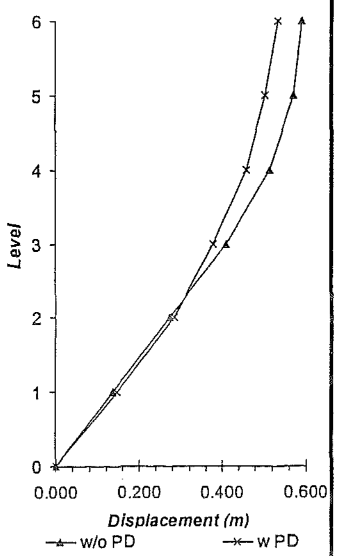
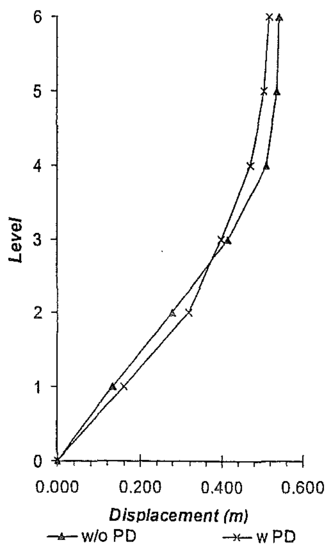
The response of structures 6/6/3101/T and 6/6/1170/T for non-linear time-history analyses when using ground acceleration records El Centro 1 and Tabas 1, scaled to represent a 2,500 years return period earthquake as described in section 5.13.3, was obtained. Tables 6.18 and 6.19 summarize the response of these analyses.

If the maximum base shear obtained using ground acceleration record El Centro 1 with a 475 years return period is compared to that using the same record but scaled to match a 2,500 years return period earthquake, an increase in the maximum base shear of 32% and 42% for structures 6/6/3101/T and 6/6/1170/T respectively is observed.

The maximum inter-storey drifts when using record El Centro 1 scaled to a 2,500 years return period earthquake are 4.3% and 4.8% for structures 6/6/3101/T and 6/6/1170/T. For inter-storey drifts of this magnitude, collapse can be avoided with proper detailing and an adequate distribution of the rotations of members. The response of both structures to ground acceleration Tabas 1 when scaled to a 2,500 years return period earthquake was satisfactory. The maximum inter-storey drifts when using this record were 3.2% and 3.6% for structures 6/6/3101/T and 6/6/1170/T.

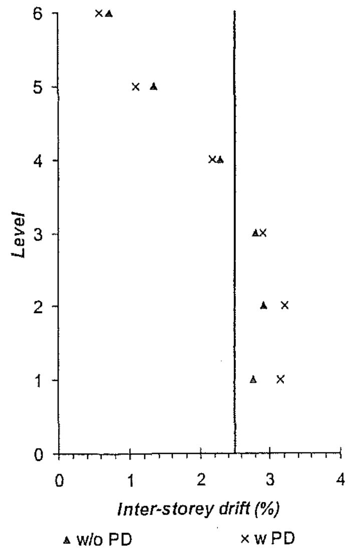
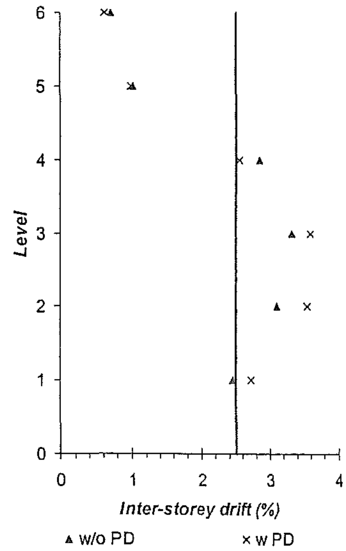
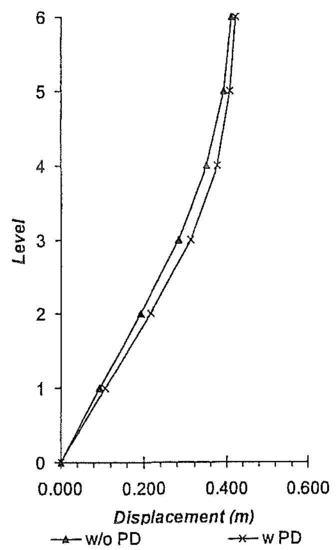
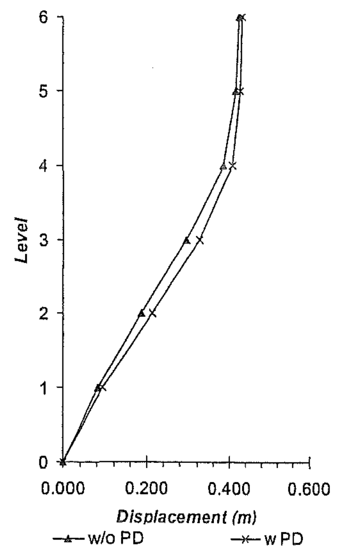
Even though the maximum inter-storey drifts of structure 6/6/1170/T were higher than those of structure 6/6/3101/T when using both records scaled to represent a 2,500 year return period earthquake, the maximum beam rotations were smaller. This difference in the beam rotations could mean the difference between collapse and survival. It is interesting to notice the difference of magnitude in beam rotations between the lower half of the building and the upper half. This is caused partly because of the additional strength given to the upper levels in order to comply with the minimum steel requirements.

**Table 6-18 Result comparison between models 6/6/3101/T and 6/6/1170/T for El Centro 1 ground acceleration record scaled to match a 2,500 years return period earthquake**

| *EL Centro 1* | | |
|---|---|---|
| | *6/6/3101/T* | *6/6/1170/T* |
| *With P-Δ* | *Maximum base shear (kN)*<br>2317.8 | *Maximum base shear (kN)*<br>2166.9 |
| *Inter-storey drifts* |  |  |
| *Maximum displacements* |  |  |

| EL Centro 1 | | |
|---|---|---|
| | *6/6/3101/T* | *6/6/1170/T* |
| *Level* | *Central span beam rotation (rad)* | *Central span beam rotation (rad)* |
| *1* | 0.0425 | 0.0349 |
| *2* | 0.0373 | 0.0317 |
| *3* | 0.0289 | 0.0248 |
| *4* | 0.0185 | 0.0092 |
| *5* | 0.0130 | 0.0005 |
| *6* | 0.0003 | 0.0001 |
| | *Interior column rotation (rad)* | *Interior column rotation (rad)* |
| *G. L.* | 0.0396 | 0.0437 |
| *1 Below* | 0.0019 | 0.0135 |
| *1 Above* | 0.0019 | 0.0200 |
| *2 Below* | 0.0136 | 0.0281 |
| *2 Above* | 0.0014 | 0.0142 |
| *3 Below* | 0.0080 | 0.0212 |
| *3 Above* | 0.0010 | 0.0071 |
| *4 Below* | 0.0094 | 0.0209 |
| *4 Above* | 0.0012 | 0.0032 |
| *5 Below* | 0.0023 | 0.0115 |
| *5 Above* | 0.0007 | 0.0033 |
| *6 Below* | 0.0111 | 0.0051 |

**Table 6-19 Result comparison between models 6/6/3101/T and 6/6/1170/T for Tabas 1 ground acceleration record scaled to match a 2,500 years return period earthquake**

| *Tabas 1* | | |
|---|---|---|
| | *6/6/3101/T* | *6/6/1170/T* |
| *With P-Δ* | *Maximum base shear (kN)* 1922.0 | *Maximum base shear (kN)* 1721.0 |
| *Inter-storey drifts* |  |  |
| *Maximum displacements* |  |  |
| *Level* | *Central span beam rotation (rad)* | *Central span beam rotation (rad)* |
| *1* | 0.0312 | 0.0242 |
| *2* | 0.0304 | 0.0271 |
| *3* | 0.0239 | 0.0259 |
| *4* | 0.0137 | 0.0063 |
| *5* | 0.0057 | 0.0005 |
| *6* | 0.0003 | 0.0001 |

| *Tabas 1* | | |
|---|---|---|
| | *6/6/3101/T* | *6/6/1170/T* |
| | ***Interior column rotation (rad)*** | ***Interior column rotation (rad)*** |
| *G. L.* | 0.0290 | 0.0251 |
| ***1 Below*** | 0.0015 | 0.0037 |
| ***1 Above*** | 0.0021 | 0.0141 |
| ***2 Below*** | 0.0017 | 0.0117 |
| ***2 Above*** | 0.0016 | 0.0119 |
| ***3 Below*** | 0.0049 | 0.0133 |
| ***3 Above*** | 0.0011 | 0.0043 |
| ***4 Below*** | 0.0056 | 0.0213 |
| ***4 Above*** | 0.0009 | 0.0075 |
| ***5 Below*** | 0.0035 | 0.0102 |
| ***5 Above*** | 0.0009 | 0.0060 |
| ***6 Below*** | 0.0049 | 0.0074 |

# 7. TWELVE STOREY BUILDING "*A*"

According to the Loadings Standard (Standards New Zealand. 1992), the maximum allowable inter-storey drift when designing using a modal method of analysis is 1.50% for a total height of the frame of 40.8 m. For non-linear time-history analyses, the maximum inter-storey drift is 2.50%.

## *7.1 Modal Analysis Results*

The dynamic properties of the 12 storey A frame are listed in Table 7.1. The square root sum of the squares combination method was used.

**Table 7-1 Dynamic properties of the 12 storey A structure**

| *Mode* | *Effective mass (Tonnes)* | *% Mass (Cumulative)* | *Period (seconds)* | $V_{max}$ *(kN)* |
|---|---|---|---|---|
| *1* | 2207.0 | 82 | 2.00 | 744.8 |
| *2* | 267.5 | 92 | 0.66 | 246.7 |
| *3* | 92.2 | 95 | 0.38 | 137.5 |
| *4* | 47.3 | 97 | 0.27 | 70.6 |
| *5* | 28.2 | 98 | 0.20 | 42.2 |
| *6* | 18.3 | 99 | 0.16 | 27.4 |
| $\Sigma=$ | 2660.7 | | | 801.2 |

**Table 7-2 Combined modal inter-storey drifts, P-delta induced forces and total inter-storey drifts for structure 12A**

| *Storey* | *Combined modal inter-storey drifts (%)* | *P-delta forces* (kN)* | *Combined modal plus P-delta inter-storey drifts (%)* |
|---|---|---|---|
| *1* | 0.77 | -59.34 | 1.17 |
| *2* | 0.96 | 53.70 | 1.50 |
| *3* | 0.93 | 62.43 | 1.45 |
| *4* | 0.88 | 59.71 | 1.33 |
| *5* | 0.82 | 56.06 | 1.19 |
| *6* | 0.75 | 51.71 | 1.06 |
| *7* | 0.68 | 47.83 | 0.93 |
| *8* | 0.60 | 41.62 | 0.79 |
| *9* | 0.51 | 34.94 | 0.65 |
| *10* | 0.42 | 27.95 | 0.51 |
| *11* | 0.31 | 18.79 | 0.37 |
| *12* | 0.20 | 8.70 | 0.23 |
| | $\Sigma=$ | *404.11* | |

**Note that P-delta forces have already been factorised by $\beta$.*

Table 7.2 shows the inter-storey drifts from the combined modal analyses, the P-delta forces factorised by the factor β, and the total inter-storey drifts which are the addition of the combined modal and P-delta inelastic inter-storey drifts.

According to New Zealand Loadings Standard (Standards New Zealand. 1992), a minimum base shear of 2.4% of the total weight of the building should be considered when designing through a modal analysis. Since 801 kN is greater than the 2.4% mentioned earlier, no modifications were made.

As noted previously for the six storey building, a minimum base shear of 3.2% is set for regular buildings in the Wellington area. This would result in a minimum base shear of 844.8 kN which is approximately a 5% increase to the combined modal base shear considered in this design. At the time that the analyses were made, the minimum base shear requirements set by the NZS 1170.5 draft had not been established, hence the base shear was not modified accordingly.

### *7.2 Non-linear Time-history Analyses Results*

Figures 7.1 and 7.2 show the top floor displacement versus time relation and the maximum inter-storey drifts for the non-linear time-history analyses with and without P-delta effects of model 12A/6/E/T using the modified ground acceleration records El Centro EW and TAFT1 respectively. P-delta effects have an important effect in the lower half of the structure. The resulting inter-storey drifts are less than the acceptable 2.5%, as defined by the Loadings Standard (Standards New Zealand. 1992). The importance of P-delta effects in the assessment of residual displacements is evident.

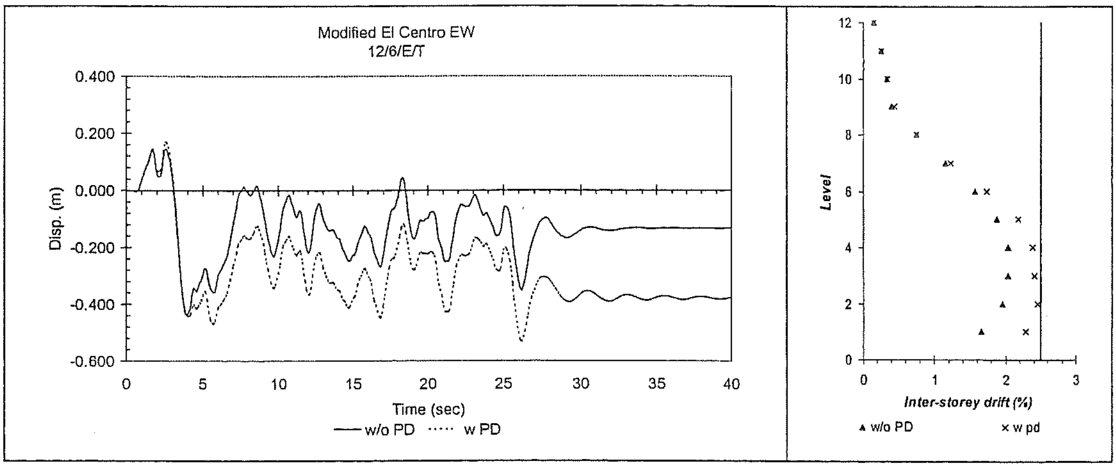



**Figure 7-1 Top floor displacement versus time and maximum inter-storey drifts for model 12A/6/E/T using ground acceleration record modified El Centro EW**

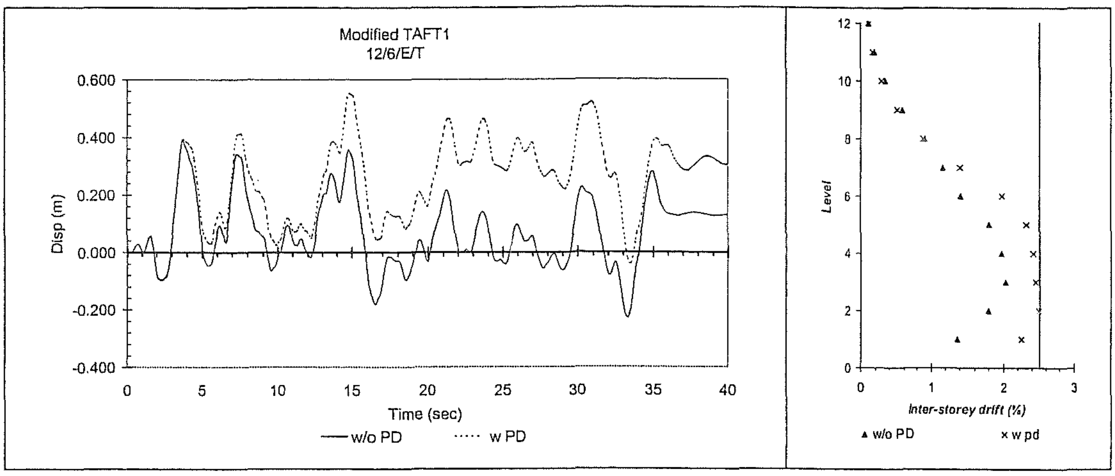



**Figure 7-2 Top floor displacement versus time and maximum inter-storey drifts for model 12A/6/E/T using ground acceleration record modified TAFT 1**

### *7.2.1 Comparison between frames modelled with a bilinear or a Takeda hysteretic rule*

Model 12A/6/E/T was reanalysed as a 12A/6/E/B model using a bilinear hysteretic rule. Non-linear time-history analyses using the four modified ground acceleration records were made for both structures to compare the maximum inter-storey drifts and maximum displacements.

Figures 7.3 and 7.4 show the maximum inter-storey drifts for the non-linear time-history analyses of structures 12A/6/E/T and 12A/6/E/B using the four modified ground acceleration records without and with P-delta effects respectively. None of the inter-storey drifts go above the 2.5% limit when P-delta effects are not considered. An acceptable maximum inter-storey drift is obtained in all cases when a Takeda hysteretic rule is used. For the modified ground acceleration records El Centro EW and TAFT 1, the maximum inter-storey drifts at the first storey when P-delta effects are considered are of the order of 2.2% for both records when a Takeda rule is used but go as high as 3.9% and 5.1% for each record respectively when a bilinear rule is used.

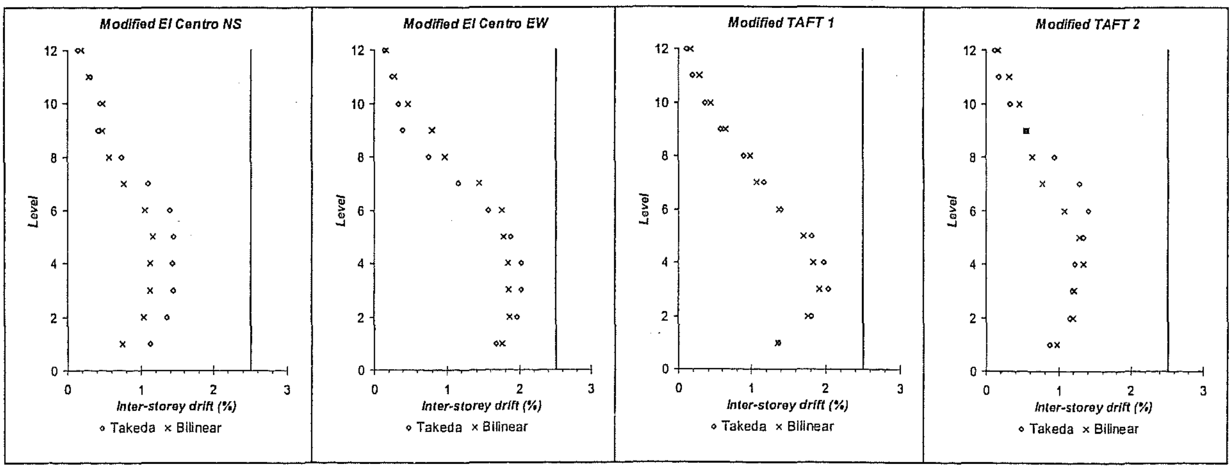



**Figure 7-3 Maximum inter-storey drifts for non-linear time-history analyses without considering P-delta effects of models 12A/6/E/T and 12A/6/E/B.**

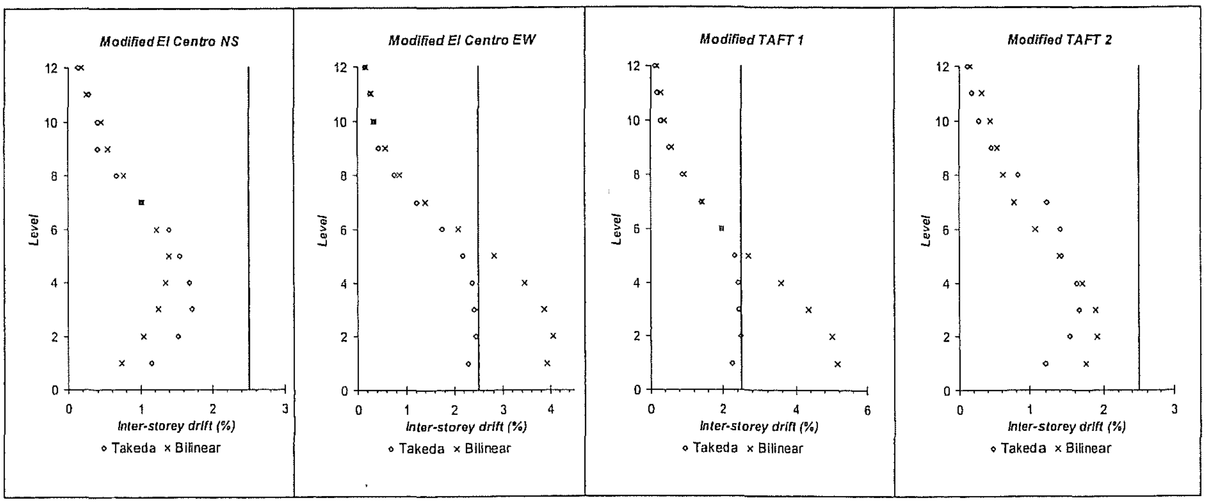



**Figure 7-4 Maximum inter-storey drifts for non-linear time-history analyses considering P-delta effects of models 12A/6/E/T and 12A/6/E/B.**

Figure 7.5 shows the averaged maximum inter-storey drifts for the four modified ground acceleration records. For these analyses, the lower levels are the most affected by the choice of hysteretic rule. Table 7.3 shows that for storeys 1 to 5, when P-delta actions are not included, the averaged maximum inter-storey drifts of the bilinear model are up to 10% greater than the corresponding values for the Takeda model. When P-delta effects are considered, the increase is of up to 30%. It should be noted that for both models, 12A/6/E/T and 12A/6/E/B, the strength of the beams from level 5 upwards is the same in order to comply with minimum steel requirements.

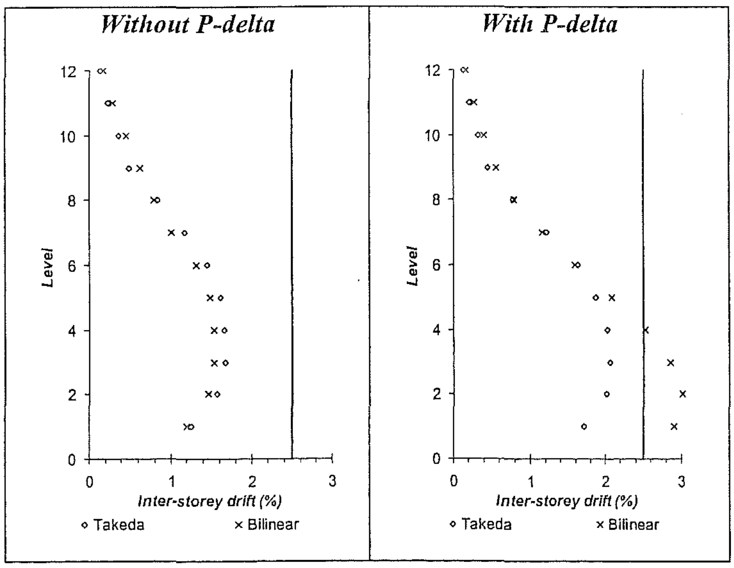



**Figure 7-5 Averaged maximum inter-storey drifts for four modified ground acceleration records to structures 12A/6/E/T and 12A/6/E/B**

**Table 7-3 Comparison of averaged maximum Inter-storey drifts of structures 12A/6/E/T and 12A/6/E/B using four modified ground acceleration records**

| | *Inter-storey drifts ratio (Takeda / Bilinear)* | |
|---|---|---|
| *Storey* | *Without P-delta* | *With P-delta* |
| *1* | 1.05 | 0.59 |
| *2* | 1.07 | 0.67 |
| *3* | 1.10 | 0.72 |
| *4* | 1.09 | 0.80 |
| *5* | 1.09 | 0.90 |
| *6* | 1.10 | 1.02 |
| *7* | 1.16 | 1.06 |
| *8* | 1.05 | 0.98 |
| *9* | 0.79 | 0.80 |
| *10* | 0.79 | 0.81 |
| *11* | 0.80 | 0.78 |
| *12* | 0.76 | 0.80 |

Figure 7.6 shows the maximum floor displacements for the four modified ground acceleration records considering P-delta effects. From the maximum displacements and maximum inter-storey drifts it can be seen that the upper levels are not contributing much to energy dissipation in any of the two structures.

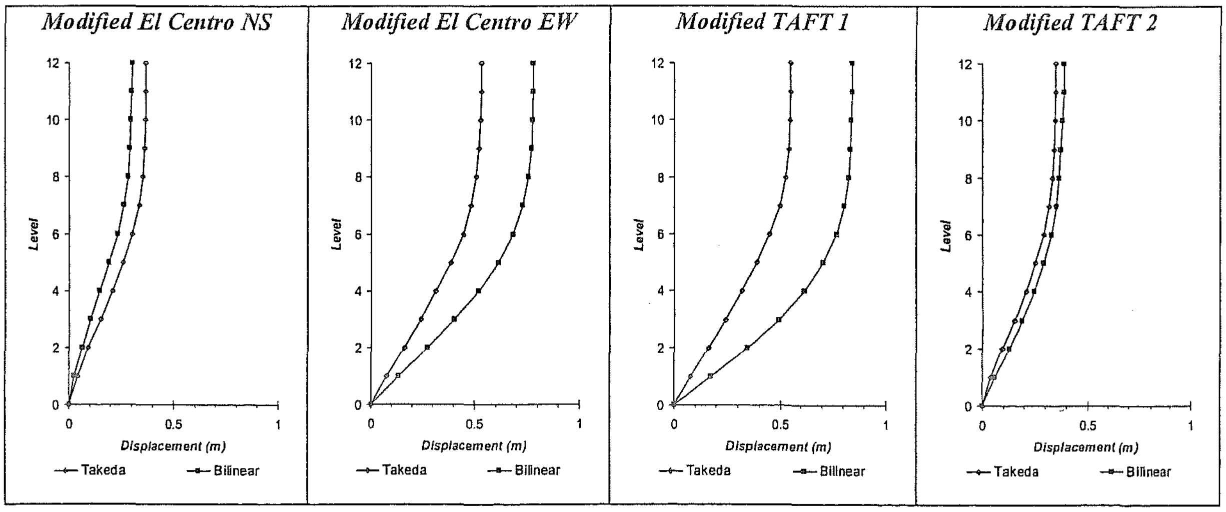



**Figure 7-6 Maximum displacements for structures 12A/6/E/T and 12A/6/E/B for four modified ground acceleration records with P-delta effects**

Figure 7.7 shows the moment versus curvature at the base of the left interior column using the modified ground acceleration record El Centro EW for structures 12A/6/E/T and 12A/6/E/B. This figure illustrates the way energy is dissipated through small cycles when a Takeda hysteretic rule is used. It also shows how once the structure is displaced in one direction, a larger force is needed to restore the structure to its original position when the bilinear hystertic rule is used compare to the corresponding force if the Takeda hysteretic rule is used. This effect is caused due to the reduction in unloading stiffness with the Takeda hysteretic model.

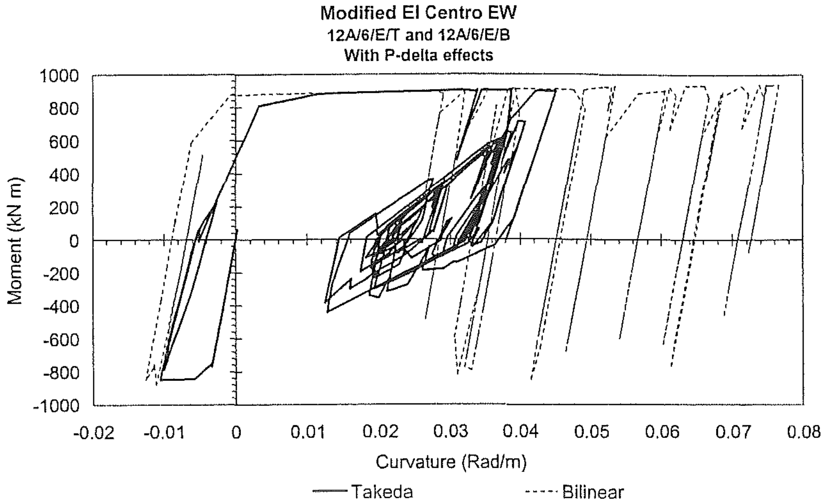



**Figure 7-7 Moment-curvature of interior column of models 12A/6/E/T and 12A/6/E/B for modified ground acceleration El Centro EW**

### *7.2.2 Axial load variation of interior columns*

Because the rotations of interior columns will be studied, analyses were made to understand their axial load variation. Table 7.4 shows the design axial load for the interior columns of model 12A/6/3101/T and a ratio between the design axial load over the maximum and the minimum axial load obtained from the non-linear time-history analyses using the four modified ground acceleration records. Since the interior columns are always in compression, the most critical value would be that from the load giving the smallest compression. A minus sign in the table indicates compression.

Table 7.5 shows the reinforcement ratio ($\rho_{design}$) of the interior columns of structure 12A/6/3101/T and the reinforcement ratio ($\rho_{max}$) if these same columns were designed using the most critical axial load value highlighted on the previous table. At level 1, this variation would represent using an area of steel of 2,950 mm$^2$ instead of 2,800 mm$^2$ which would translate in one 13.8 mm diameter bar. The variation between these values can be neglected for all practical purposes.

**Table 7-4 Ratio between design axial load in columns over maximum and minimum column axial load values from non-linear time-history analyses for model 12A/6/3101/T**

| *Column design axial load over maximum and minimum axial load from time-history analyses* | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| *Level* | *Design values (kN)* | *El Centro NS* | | *EL Centro EW* | | *TAFT 1* | | *TAFT 2* | | |
| | | *Max* | *Min* | *Max* | *Min* | *Max* | *Min* | *Max* | *Min* | |
| 1 | -1401.9 | 1.06 | 0.95 | 1.06 | 0.96 | 1.06 | 0.95 | ***1.07*** | 0.96 | |
| 2 | -1276.5 | 1.05 | 0.97 | 1.05 | 0.97 | 1.05 | 0.96 | ***1.07*** | 0.97 | |
| 3 | -1151.0 | 1.04 | 0.96 | 1.05 | 0.98 | 1.05 | 0.96 | ***1.06*** | 0.97 | |
| 4 | -1025.6 | 1.04 | 0.96 | ***1.06*** | 0.98 | 1.05 | 0.97 | ***1.06*** | 0.97 | |
| 5 | -900.2 | 1.05 | 0.98 | ***1.07*** | 0.98 | 1.06 | 0.97 | ***1.07*** | 0.97 | |
| 6 | -774.8 | 1.06 | 0.97 | ***1.07*** | 0.98 | ***1.07*** | 0.97 | ***1.07*** | 0.95 | |
| 7 | -649.37 | 1.07 | 0.97 | 1.08 | 0.98 | 1.07 | 0.97 | ***1.09*** | 0.97 | |
| 8 | -538.78 | 1.08 | 0.96 | ***1.10*** | 0.97 | 1.08 | 0.96 | ***1.10*** | 0.97 | |
| 9 | -428.19 | 1.09 | 0.96 | ***1.12*** | 0.96 | 1.10 | 0.96 | ***1.12*** | 0.97 | |
| 10 | -317.6 | 1.12 | 0.95 | ***1.13*** | 0.94 | 1.12 | 0.96 | ***1.13*** | 0.96 | |
| 11 | -207.0 | 1.16 | 0.94 | 1.15 | 0.92 | 1.16 | 0.96 | ***1.17*** | 0.93 | |
| 12 | -96.4 | 1.18 | 0.93 | 1.15 | 0.92 | ***1.22*** | 0.95 | 1.17 | 0.93 | |

**Table 7-5 Interior column reinforcement ratio variation using the most critical value for the four modified ground acceleration records**

| *Level* | $\rho_{design}$ | $\rho_{max}$ | $\rho_{max} / \rho_{design}$ |
|---|---|---|---|
| **1** | 0.0115 | 0.0121 | 1.05 |
| **2** | 0.0137 | 0.0142 | 1.04 |
| **3** | 0.0158 | 0.0162 | 1.03 |
| **4** | 0.0164 | 0.168 | 1.02 |
| **5** | 0.0150 | 0.0154 | 1.03 |
| **6** | 0.0162 | 0.0166 | 1.02 |
| **7** | 0.0174 | 0.0178 | 1.02 |
| **8** | 0.0189 | 0.0193 | 1.02 |
| **9** | 0.0203 | 0.0207 | 1.02 |
| **10** | 0.0226 | 0.0228 | 1.01 |
| **11** | 0.0168 | 0.0170 | 1.01 |
| **12** | 0.008 | 0.008 | 1.00 |

*7.2.3 Comparison between frames with different column characteristic*

A summary of the results from models 12A/6/E/T, 12A/6/3101/T and 12A/6/1170/T analysed using the 8 earthquake records described in section 5.13.2 is presented. The main characteristics of these structures are stated below. For more details on the properties of the structures, see section 5.1 and appendix A.

1. Structure 12A/6/E/T
    - Designed to a ductility of 6
    - All beams comply with minimum steel requirements
    - From level 5 upwards, beams are modelled with the same strength
    - Columns are elastic except at the base
    - A Takeda hysteretic model is used for all elements

2. Structure 12A/6/3101/T
    - Designed to a ductility of 6
    - All beams comply with minimum steel requirements
    - From level 5 upwards, beams are modelled with the same strength
    - Columns are designed to NZS 3101:1995 (Standards New Zealand. 1995)
    - A Takeda hysteretic model is used for all elements

3. Structure 12A/6/1170/T
    - Designed to a ductility of 6
    - All beams comply with minimum steel requirements.
    - From level 5 upwards, beams are modelled with the same strength
    - Columns are designed to NZS 1170.5 draft for limited protection against plastic hinges
    - A Takeda hysteretic model is used for all elements

Appendix B, part two, shows the response of structures 12A/6/E/T, 12A/6/3101/T and 12A/6/1170/T when subject to each of the selected ground acceleration records. The same results as for the six storey structures are reported.

When running the non-linear time-history analyses for structure 12A/6/1170/T, problems with the structural analysis program Ruaumoko (Carr 2003) were encountered. For most cases, by adding the terms corresponding to the rotational degrees of freedom to the mass matrix these problems were overcame.

Structure 12A/6/1170/T could not be analysed using ground acceleration record Tabas 2. The structural analysis program (Carr 2003) gave a message error *"Not a number in residual displacements"*. This was probably caused by an underflow. Alterations are being made to the program to overcome this problem, but due to time limitations, the results using this record could not be included in this work.

*7.2.3.1 **Base Shear***

Table 7.6 shows the average maximum base shear and standard deviation for the non-linear time-history analyses of structures 12A/6/E/T, 12A/6/3101/T and 12A/6/1170/T. It also gives the equivalent static and modal base shear and the P-delta forces used for each of these analyses. The difference between the averaged maximum base shear of the three structures in the time-history analyses is less than 1% when P-delta effects are not considered and of 5% when they are considered. Structures 12A/6/E/T and 12A/6/3101/T differ in their average maximum base shears by less than 1% with and without P-delta effects.

**Table 7-6 Base shear summary for structures 12A/6/E/T, 12A/6/3101/T and 12A/6/1170/T**

| | Eq. Static (kN) | Modal (kN) |
|---|---|---|
| ***Base shear*** | 908.2 | 801.2 |
| ***P-Δ induced**** | 392.5 | 404.1 |
| ***Σ*=** | 1300.7 | 1205.3 |

| | 12A/6/E/T (kN) | | 12A/6/3101/T (kN) | | 12A/6/1170/T (kN) | |
|---|---|---|---|---|---|---|
| | **Average** | **Std. Dev.** | **Average** | **Std. Dev.** | **Average** | **Std. Dev.** |
| ***Without PD*** | 1889.4 | 77.3 | 1887.9 | 79.3 | 1870.3 | 59.4 |
| ***With PD*** | 2151.3 | 180.1 | 2147.1 | 172.2 | 2057.1 | 92.5 |

**P-delta forces are already multiplied by the factor β.*

There is an increase of around two fold when comparing the base shear from the modal analysis with P-delta to the averaged maximum base shear from the non-linear time-history analyses of the 3 structures. Appendix C shows the influence that damping forces have on the maximum base shear values from the non-linear time-history analyses. The non-linear time-history analyses show an increase in the averaged maximum base shear of 14% for structures 12A/6/E/T and 12A/6/3101/T when P-delta effects are considered compared to the case where they are not. For structure 12A/6/1170/T this increase is of 10%. For the elastic analyses, the base shear was increased by up to 50% to account for P-delta effects.

If the average maximum base shear from the non-linear time-history analyses using the modified ground acceleration records is compared to that using the natural records, the difference for the 3 structures is 2% or less.

*7.2.3.2 Inter-storey drifts*

Table 7.7 shows the averaged maximum inter-storey drifts and its standard deviation for the non-linear time-history analyses of structures 12A/6/E/T, 12A/6/3101/T and 12A/6/1170/T considering P-delta effects. The average maximum inter-storey drifts of structures 12A/6/E/T and 12A/6/3101/T do not differ at any level by a margin greater than 1%. None of the averaged maximum inter-storey drifts exceed the allowable 2.5%. The greatest difference between structures 12A/6/3101/T and 12A/6/1170/T arises at level 4 which coincides with the maximum averaged inter-storey drifts of both structures.

**Table 7-7 Average maximum inter-storey drifts and standard deviations considering P-delta effects using eight ground acceleration records for structures 12A/6/E/T, 12A/6/3101/T and 12A/6/1170/T**

| | | *12A/6/E/T* | | *12A/6/3101/T* | | *12A/6/1170/T* | |
|---|---|---|---|---|---|---|---|
| | | *Avg.* | *Std. dev.* | *Avg.* | *Std. dev.* | *Avg.* | *Std. dev.* |
| *With P-delta* | *Storey* | *(%)* | *(%)* | *(%)* | *(%)* | *(%)* | *(%)* |
| | *1* | 1.33 | 0.60 | 1.31 | 0.58 | 1.02 | 0.64 |
| | *2* | 1.58 | 0.59 | 1.59 | 0.61 | 1.73 | 1.15 |
| | *3* | 1.69 | 0.52 | 1.70 | 0.54 | 2.14 | 1.56 |
| | *4* | 1.69 | 0.51 | 1.69 | 0.52 | 2.23 | 1.69 |
| | *5* | 1.59 | 0.46 | 1.59 | 0.47 | 2.00 | 1.45 |
| | *6* | 1.39 | 0.35 | 1.39 | 0.34 | 1.30 | 0.75 |
| | *7* | 1.05 | 0.22 | 1.04 | 0.22 | 0.80 | 0.44 |
| | *8* | 0.68 | 0.14 | 0.68 | 0.14 | 0.49 | 0.23 |
| | *9* | 0.42 | 0.06 | 0.42 | 0.06 | 0.32 | 0.14 |
| | *10* | 0.32 | 0.06 | 0.32 | 0.06 | 0.28 | 0.14 |
| | *11* | 0.22 | 0.05 | 0.22 | 0.05 | 0.20 | 0.09 |
| | *12* | 0.14 | 0.02 | 0.13 | 0.02 | 0.13 | 0.05 |

Table 7.8 shows the increase in the averaged maximum inter-storey drifts when P-delta effects are considered in the non-linear time-history analyses to the corresponding values when they are not. Structures 12A/6/E/T and 12A/6/3101/T show a 23% and 22% increase respectively in their averaged maximum inter-storey drifts while structure 12A/6/1170/T show an increase of up to 51% when P-delta effects are considered in the analyses. For the lower half of the three structures, P-delta effects make the averaged maximum inter-storey drifts increase while the upper half shows a reduction of up to 8%.

Figure 7.8 illustrates the scatter of the maximum inter-storey drifts from the non-linear time-history analyses to structures 12A/6/3101/T and 12A/6/1170/T when P-delta effects are considered. Structure 12A/6/3101/T complies in all records with the maximum allowable inter-storey drift of 2.5% just being slightly above it when ground acceleration record TAFT 1 was used. Structure 12A/6/1170/T showed critical maximum inter-storey drift values when records Modified El Centro EW

**Table 7-8 Comparison between the maximum average inter-storey drifts with P-delta and without P-delta for structures 12A/6/E/T, 12A/6/3101/T and 12A/6/1170/T**

| *Averaged maximum Inter-storey drifts With P-delta / Without P-delta* | | | | | | | |
|---|---|---|---|---|---|---|---|
| *Storey* | *12A/6/E/T* | *12A/6/3101/T* | *12A/6/1170/T* | *Storey* | *12A/6/E/T* | *12A/6/3101/T* | *12A/6/1170/T* |
| *1* | 1.23 | 1.22 | 1.23 | *7* | 0.97 | 0.97 | 0.93 |
| *2* | 1.17 | 1.17 | 1.30 | *8* | 0.92 | 0.92 | 0.92 |
| *3* | 1.15 | 1.16 | 1.46 | *9* | 0.92 | 0.92 | 0.94 |
| *4* | 1.12 | 1.13 | 1.51 | *10* | 0.90 | 0.90 | 0.97 |
| *5* | 1.08 | 1.08 | 1.40 | *11* | 0.96 | 0.96 | 0.97 |
| *6* | 1.05 | 1.04 | 1.03 | *12* | 1.00 | 1.00 | 1.00 |

and Modified TAFT 1 were used resulting in inter-storey drifts of up to 3.8% and 5.5%. For the other 5 records the results are satisfactory. The averaged maximum inter-storey drifts for the non-linear time-history analyses using the modified records, compared to the corresponding values when the natural records are used show an increase in the averaged results of 84% and 280% for structures 12A/6/3101/T and 12A/6/1170/T.

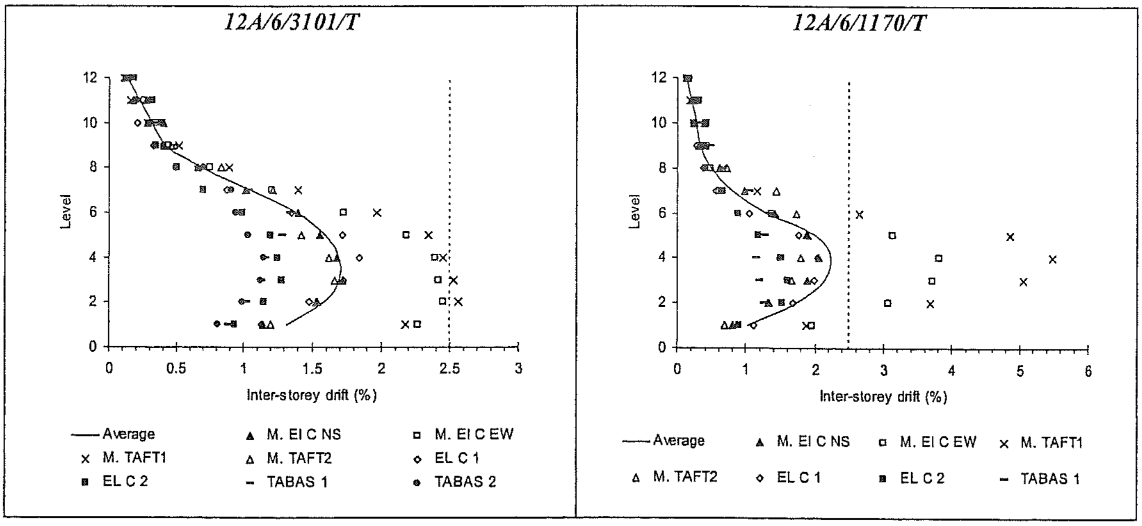



**Figure 7-8 Maximum inter-storey drifts for non-linear time-history analyses of structures 12A/6/3101/T and 12A/6/1170/T**

### *7.2.3.3 Maximum displacements*

Table 7.9 shows the averaged displacements and standard deviations from the non-linear time-history analyses of structures 12A/6/E/T, 12A/6/3101/T and 12A/6/1170/T considering P-delta effects.

The results from structures 12A/6/E/T and 12A/6/3101/T show no significant variation. The averaged maximum displacements when P-delta is considered vary from the corresponding values without considering P-delta in 23% as for the averaged maximum inter-storey drifts.

Figure 7.9 illustrates the maximum displacements from the time-history non-linear analyses of structures 12A/6/3101/T and 12A/6/1170/T when P-delta effects are considered, along with the deformed shape coming from the modal analysis and P-delta actions. Except for records Modified El Centro EW and TAFT 1, the maximum displacements match well those displacements calculated using the modal analysis and P-delta. If the averaged maximum displacements are compared to the modal and P-delta displacements, the difference is considerable but this is related to the high standard deviation values for the 3 structures. This is clearly illustrated in Figure 7.9. Appendix D shows the displacement response spectra for the ground acceleration records used.

**Table 7-9 Averaged maximum displacements and standard deviations for non-linear time-history analyses of structures 12A/6/E/T, 12A/6/3101/T and 12A/6/1170/T considering P-delta effects**

| Level | *12A/6/E/T* | | *12A/6/3101/T* | | *12A/6/1170/T* | |
|---|---|---|---|---|---|---|
| | Average (m) | Std Deviation (m) | Average (m) | Std Deviation (m) | Average (m) | Std Deviation (m) |
| 1 | 0.045 | 0.019 | 0.045 | 0.018 | 0.040 | 0.017 |
| 2 | 0.098 | 0.038 | 0.098 | 0.038 | 0.105 | 0.047 |
| 3 | 0.153 | 0.055 | 0.153 | 0.056 | 0.183 | 0.091 |
| 4 | 0.208 | 0.069 | 0.208 | 0.070 | 0.268 | 0.140 |
| 5 | 0.259 | 0.083 | 0.260 | 0.084 | 0.343 | 0.181 |
| 6 | 0.301 | 0.095 | 0.301 | 0.095 | 0.391 | 0.194 |
| 7 | 0.331 | 0.103 | 0.331 | 0.104 | 0.417 | 0.196 |
| 8 | 0.348 | 0.109 | 0.348 | 0.109 | 0.429 | 0.197 |
| 9 | 0.357 | 0.111 | 0.357 | 0.112 | 0.434 | 0.197 |
| 10 | 0.361 | 0.112 | 0.361 | 0.112 | 0.437 | 0.196 |
| 11 | 0.364 | 0.113 | 0.364 | 0.113 | 0.438 | 0.196 |
| 12 | 0.365 | 0.113 | 0.366 | 0.113 | 0.440 | 0.197 |

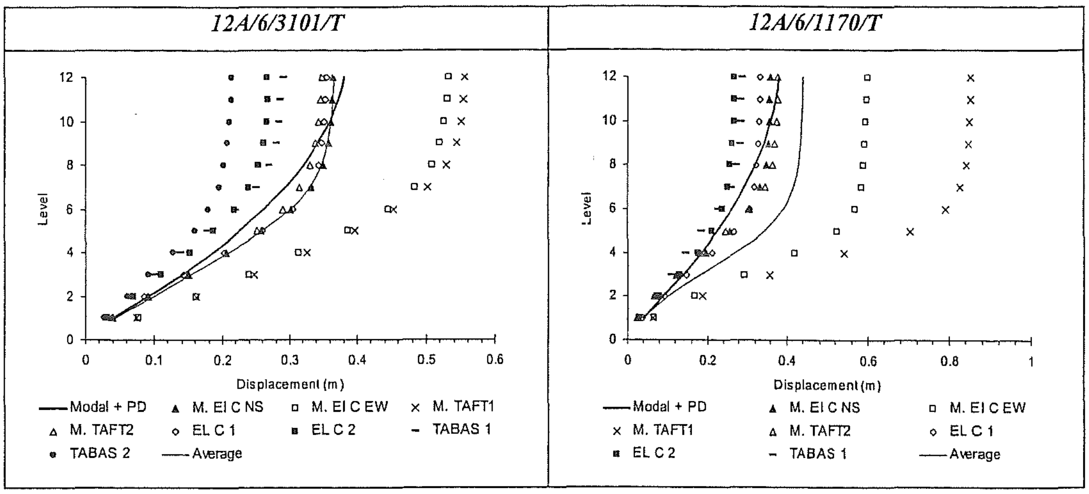



**Figure 7-9 Maximum displacements for non-linear time-history analyses of structures 12A/6/3101/T and 12A/6/1170/T**

### *7.2.3.4 Beam rotations*

The rotations from the non-linear time-history analyses and the rotations for the equivalent static and modal analyses were calculated as described in section 6.3.4.4. Table 7.10 shows the averaged maximum beam rotations from the non-linear time-history analyses and its standard deviation of structures 12A/6/E/T, 12A/6/3101/T and 12A/6/1170/T. It also shows the design total rotations from the equivalent static and modal analyses calculated as described in section 6.3.4.1. Figure 7.10 shows the average maximum beam rotation values for the three structures and the total beam rotation values from the equivalent static and modal analyses from table 7.10. Structures 12A/6/E/T and 12A/6/3101/T show a small difference in the averaged maximum beam rotations.

**Table 7-10 Equivalent static and modal analysis beam rotation demand and average of maximum beam rotations and standard deviations from non-linear time-history analyses of structures 12A/6/E/T, 12A/6/3101/T and 12A/6/1170/T**

| *Level* | *Eq. static rotations* (rad)* | *Modal rotations* (rad)* | *12A/6/E/T* | | *12A/6/3101/T* | | *12A/6/1170/T* | |
|---|---|---|---|---|---|---|---|---|
| | | | *Average rot. (rad)* | *Std. Dev. (rad)* | *Average rot. (rad)* | *Std. Dev. (rad)* | *Average rot. (rad)* | *Std. Dev. (rad)* |
| *1* | 0.0191 | 0.0196 | 0.0136 | 0.0066 | 0.0135 | 0.0063 | 0.0105 | 0.0053 |
| *2* | 0.0249 | 0.0250 | 0.0157 | 0.0060 | 0.0159 | 0.0063 | 0.0161 | 0.0071 |
| *3* | 0.0247 | 0.0240 | 0.0163 | 0.0056 | 0.0164 | 0.0058 | 0.0187 | 0.0091 |
| *4* | 0.0233 | 0.0220 | 0.0161 | 0.0054 | 0.0161 | 0.0055 | 0.0147 | 0.0068 |
| *5* | 0.0216 | 0.0198 | 0.0147 | 0.0046 | 0.0147 | 0.0046 | 0.0119 | 0.0042 |
| *6* | 0.0197 | 0.0177 | 0.0117 | 0.0031 | 0.0117 | 0.0030 | 0.0081 | 0.0032 |
| *7* | 0.0177 | 0.0155 | 0.0075 | 0.0019 | 0.0074 | 0.0019 | 0.0046 | 0.0017 |
| *8* | 0.0154 | 0.0133 | 0.0038 | 0.0011 | 0.0038 | 0.0011 | 0.0020 | 0.0008 |
| *9* | 0.0131 | 0.0111 | 0.0020 | 0.0006 | 0.0020 | 0.0007 | 0.0017 | 0.0009 |
| *10* | 0.0107 | 0.0089 | 0.0010 | 0.0007 | 0.0010 | 0.0007 | 0.0011 | 0.0007 |
| *11* | 0.0082 | 0.0066 | 0.0004 | 0.0001 | 0.0004 | 0.0001 | 0.0004 | 0.0000 |
| *12* | 0.0057 | 0.0044 | 0.0002 | 0.0000 | 0.0002 | 0.0000 | 0.0002 | 0.0000 |

**Scaled beam rotations by factor f, as described in section 7.3.4.1 to match the allowable 2.5% inter-storey drift limit for the non-linear analysis*

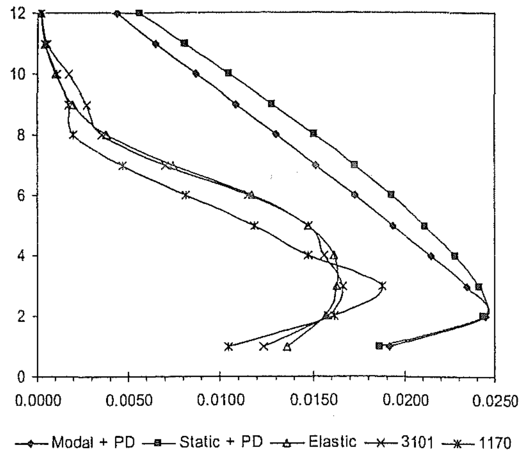
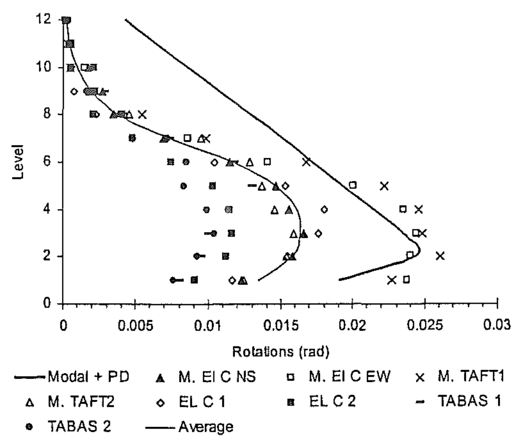



**Figure 7-10 Equivalent static and modal analysis beam rotation demand and average of maximum beam rotations from non-linear time-history analyses of structures 12A/6/E/T, 12A/6/3101/T and 12A/6/1170/T**

Figure 7.11 shows the scatter of the maximum rotations from the non-linear time-history analyses of structures 12A/6/3101/T and 12A/6/1170/T. The beam rotation values are only greater than the modal rotations values for records Modified El Centro EW and Modified TAFT 1 for both structures. The maximum inter-storey drifts of structure 12A/6/3101/T for these two records were close to 2.5% so that it was expected for rotations to match with the modal rotations is satisfactory.

*12A/6/3101/T*




*12A/6/1170/T*

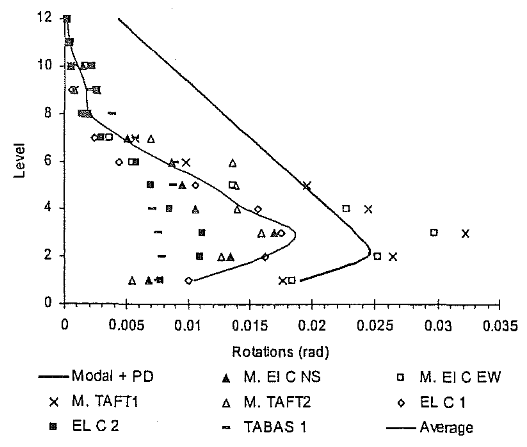



**Figure 7-11 Maximum beam rotations of structures 12A/6/3101/T and 12A/6/1170/T from non-linear time-history analyses**

Table 7.11 shows the averaged maximum beam section ductilities for structures 12A/6/E/T, 12A/6/3101/T and 12A/6/1170/T from the non-linear time-history analyses and the beam ductility demand from the equivalent static and modal analyses. For the three structures the maximum beam ductility is close to 25 while for the equivalent static and modal analysis the maximum beam ductility is around 30.

**Table 7-11 Design beam section ductilities and averaged maximum beam section ductilities for structures 12A/6/E/T, 12A/6/3101/T and 12A/6/1170/T**

| Level | Eq *static* | Modal | 12A/6/E/T | 12A/6/3101/T | 12A/6/1170/T |
|---|---|---|---|---|---|
| | $S_\mu$ | $S_\mu$ | $S_\mu$ | $S_\mu$ | $S_\mu$ |
| 1 | 24.9 | 25.5 | 17.7 | 17.5 | 13.7 |
| 2 | 31.3 | 31.4 | 19.7 | 19.9 | 20.2 |
| 3 | 33.5 | 32.6 | 22.1 | 22.2 | 25.4 |
| 4 | 34.8 | 32.8 | 24.1 | 24.1 | 22.0 |
| 5 | 35.9 | 32.9 | 24.5 | 24.4 | 19.7 |
| 6 | 32.8 | 29.3 | 19.5 | 19.4 | 13.5 |
| 7 | 29.3 | 25.8 | 12.4 | 12.3 | 7.7 |
| 8 | 25.6 | 22.0 | 6.3 | 6.3 | 3.4 |
| 9 | 21.8 | 18.4 | 3.3 | 3.3 | 2.8 |
| 10 | 17.7 | 14.8 | 1.7 | 1.7 | 1.8 |
| 11 | 13.7 | 11.0 | 0.7 | 0.7 | 0.7 |
| 12 | 9.5 | 7.4 | 0.4 | 0.4 | 0.3 |

#### *7.2.3.5 Interior column rotations*

Column rotations were calculated as described in section 6.3.4.5. Table 7.12 shows the column yield rotation, $\theta_y$, the averaged maximum column rotation for the non-linear time-history analyses, its standard deviation and the averaged maximum section curvature ductility for an interior column of structures 12A/6/3101/T and 12A/6/1170/T respectively. The averaged maximum column section ductilities are 1.4 and 9.9 for each of the structures respectively. These values agree with the design objectives of high protection against plastic hinging in columns for structure 12A/6/3101/T and a lower level for structure 12A/6/1170/T. As mentioned before, all the rotations were calculated considering P-delta effects in the analyses.

Figure 7.12 shows the maximum rotations above and below the beam for structure 12A/6/3101/T. For the column sections above the beam only in very few cases was the yield rotation exceeded. The maximum column section rotation ductility above the beams was 2.6 at level one, while at level 2 and 3 the maximum value was 1.2. All the other levels remained well below the yield value. For the column sections below the beam, a maximum rotation ductility of 1.2 was obtained at level 7.

**Table 7-12 Interior columns design yield rotation, averaged maximum rotation and standard deviation for non-linear time-history analyses of structures 12A/6/3101/T and 12A/6/1170/T**

| *level* | *12A/6/3101/T* | | | | *12A/6/1170/T* | | | |
|---|---|---|---|---|---|---|---|---|
| | $\theta_y$ *(rad)* | *Avg. rot (rad)* | *Std. dev. (rad)* | $S_\mu$ | $\theta_y$ *(rad)* | *Avg. rot (rad)* | *Std. dev. (rad)* | $S_\mu$ |
| *G.L.* | 0.0014 | 0.0092 | 0.0046 | 6.6 | 0.0014 | 0.0086 | 0.0042 | 6.2 |
| *1 Below* | 0.0014 | 0.0009 | 0.0001 | 0.6 | 0.0014 | 0.0012 | 0.0004 | 0.9 |
| *1 Above* | 0.0014 | 0.0019 | 0.0007 | 1.4 | 0.0013 | 0.0107 | 0.0058 | 7.9 |
| *2 Below* | 0.0014 | 0.0013 | 0.0001 | 0.9 | 0.0013 | 0.0062 | 0.0040 | 4.6 |
| *2 Above* | 0.0014 | 0.0012 | 0.0001 | 0.9 | 0.0013 | 0.0096 | 0.0089 | 7.2 |
| *3 Below* | 0.0014 | 0.0013 | 0.0001 | 0.9 | 0.0013 | 0.0079 | 0.0069 | 5.9 |
| *3 Above* | 0.0014 | 0.0012 | 0.0001 | 0.9 | 0.0013 | 0.0089 | 0.0089 | 6.9 |
| *4 Below* | 0.0014 | 0.0013 | 0.0001 | 0.9 | 0.0013 | 0.0128 | 0.0105 | 9.9 |
| *4 Above* | 0.0014 | 0.0011 | 0.0002 | 0.8 | 0.0014 | 0.0103 | 0.0079 | 7.6 |
| *5 Below* | 0.0014 | 0.0013 | 0.0002 | 1.0 | 0.0014 | 0.0134 | 0.0115 | 9.8 |
| *5 Above* | 0.0014 | 0.0009 | 0.0001 | 0.6 | 0.0013 | 0.0049 | 0.0027 | 3.6 |
| *6 Below* | 0.0014 | 0.0014 | 0.0002 | 1.0 | 0.0013 | 0.0080 | 0.0050 | 5.9 |
| *6 Above* | 0.0014 | 0.0008 | 0.0001 | 0.6 | 0.0013 | 0.0017 | 0.0005 | 1.3 |
| *7 Below* | 0.0013 | 0.0014 | 0.0002 | 1.0 | 0.0013 | 0.0045 | 0.0018 | 3.4 |
| *7 Above* | 0.0013 | 0.0008 | 0.0001 | 0.6 | 0.0012 | 0.0011 | 0.0005 | 0.9 |
| *8 Below* | 0.0013 | 0.0012 | 0.0001 | 0.9 | 0.0012 | 0.0027 | 0.0011 | 2.2 |
| *8 Above* | 0.0013 | 0.0008 | 0.0001 | 0.6 | 0.0013 | 0.0012 | 0.0005 | 1.0 |
| *9 Below* | 0.0013 | 0.0010 | 0.0001 | 0.8 | 0.0013 | 0.0011 | 0.0004 | 0.8 |
| *9 Above* | 0.0013 | 0.0007 | 0.0001 | 0.5 | 0.0012 | 0.0007 | 0.0001 | 0.6 |
| *10 Below* | 0.0013 | 0.0009 | 0.0001 | 0.7 | 0.0012 | 0.0009 | 0.0002 | 0.7 |
| *10 Above* | 0.0013 | 0.0006 | 0.0001 | 0.4 | 0.0012 | 0.0006 | 0.0001 | 0.5 |
| *11 Below* | 0.0012 | 0.0007 | 0.0001 | 0.6 | 0.0012 | 0.0007 | 0.0002 | 0.6 |
| *11 Above* | 0.0012 | 0.0004 | 0.0001 | 0.3 | 0.0012 | 0.0004 | 0.0001 | 0.3 |
| *12 Below* | 0.0014 | 0.0005 | 0.0001 | 0.3 | 0.0011 | 0.0005 | 0.0001 | 0.5 |

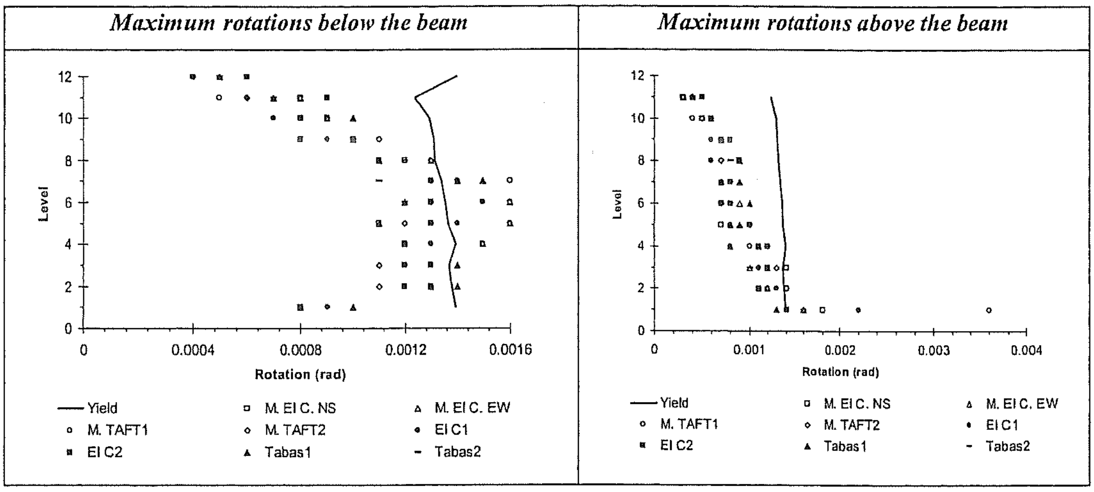



**Figure 7-12 Column maximum rotations above and below the beam for non-linear time-history analyses of structure 12A/6/3101/T**

Figure 7.13 shows the maximum rotations above and below the beam for structure 12A/6/1170/T. The maximum section rotation ductility below the beam is of 26 at levels 4 and 5 for modified ground acceleration record TAFT 1. Below the beam, the maximum column rotation ductility is 21 at levels 2 and 3 for the modified record TAFT 1.

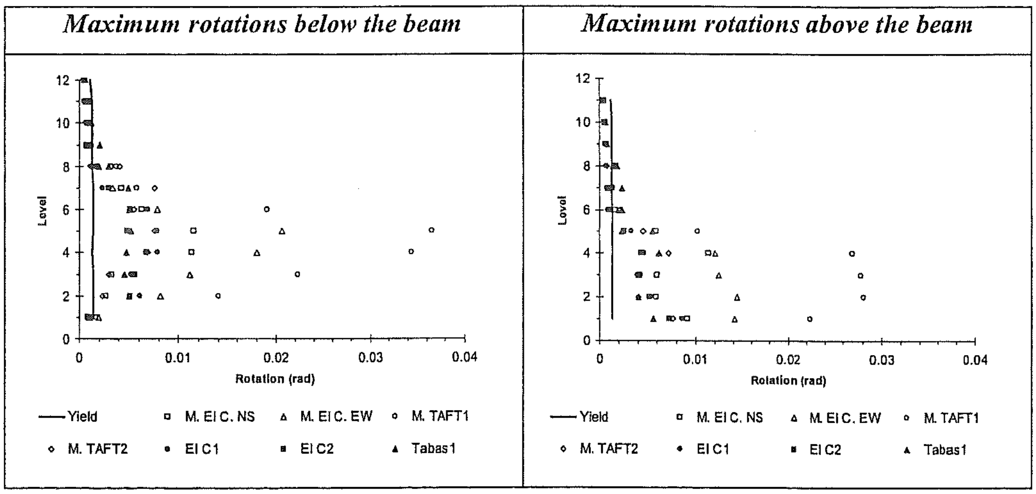



**Figure 7-13 Column maximum rotations above and below the beam for non-linear time-history analyses of structure 12A/6/1170/T**

### *7.2.4 Earthquake return period of 2,500 years*

Figure 7.14 shows the top floor time displacement obtained with the non-linear time-history analyses of structure12A/6/3101/T with and without P-delta effects, and structure 12A/6/1170/T with P-delta effects. When P-delta effects are not considered, structure 12A/6/3101/T shows no collapse with a maximum inter-storey drift of 3.6%. If P-delta effects are included, both structures develop a column-sway mechanism on the ground floor that leads them to collapse.

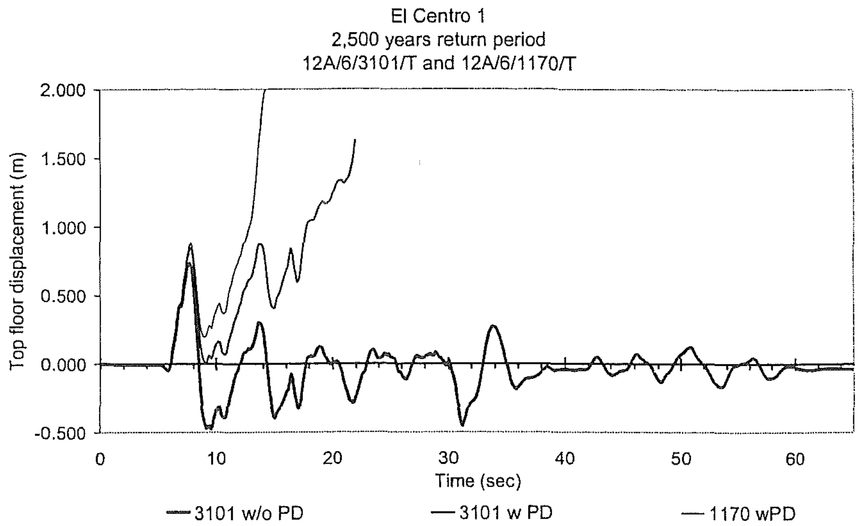



**Figure 7-14 Top floor displacement versus time from non-linear time-history analyses of structures 12A/6/3101/T and 12A/6/1170/T using 2,500 years return period El Centro 1**

Table 7.13 shows the results from the non-linear time-history analysis of structure 12A/6/3101/T using the ground acceleration record Tabas 1 scaled to a 2,500 years return period earthquake. Structure 12A/6/1170/T could not be run using this ground acceleration record. The same problem arose in the structural analysis program as occurred in the analyses with the Tabas 2 record scaled to a 475 years return period earthquake (see section 6.2.3).

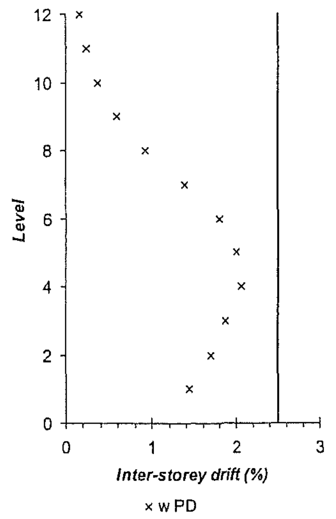
The maximum inter-storey drift when using the ground acceleration record Tabas 1 scaled to a 2,500 years return period event, for structure 12A/6/3101/T is 2%. The highest beam rotation ductility is 31 at levels 4 and 5. The base of the column reached a section rotation ductility of 8.5 and in the upper levels a section rotation ductility of 1.3. The structure would have avoided collapse with still a good strength reserve.

Table 7-13 Results for non-linear time-history analysis using record Tabas 1 with a 2,500 years return period of structure 12A/6/3101/T with P-delta effects

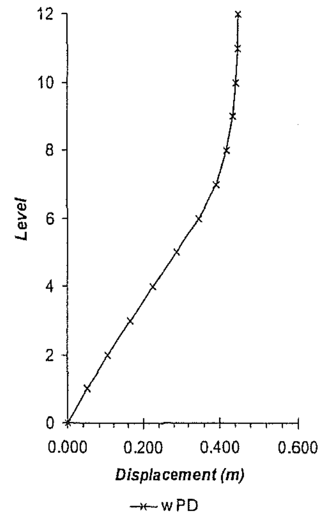
*Tabas 1*

*12A/6/3101/T*

*Inter-storey drifts*




*Maximum displacements*




| *Level* | *Central span beam rotations (rad)* | *Level* | *Central span beam rotations (rad)* |
|---|---|---|---|
| *1* | 0.0148 | *7* | 0.0099 |
| *2* | 0.0172 | *8* | 0.0065 |
| *3* | 0.0192 | *9* | 0.0032 |
| *4* | 0.0210 | *10* | 0.0008 |
| *5* | 0.0191 | *11* | 0.0005 |
| *6* | 0.0161 | *12* | 0.0003 |
| | ***Interior column rotation (rad)*** | | ***Interior column rotation (rad)*** |
| *G.L.* | 0.0119 | *6 Above* | 0.0011 |
| *1 Below* | 0.0009 | *7 Below* | 0.0017 |
| *1 Above* | 0.0016 | *7 Above* | 0.0009 |
| *2 Below* | 0.0016 | *8 Below* | 0.0013 |
| *2 Above* | 0.0014 | *8 Above* | 0.0008 |
| *3 Below* | 0.0017 | *9 Below* | 0.0013 |
| *3 Above* | 0.0014 | *9 Above* | 0.0007 |
| *4 Below* | 0.0015 | *10 Below* | 0.0010 |
| *4 Above* | 0.0011 | *10 Above* | 0.0007 |
| *5 Below* | 0.0014 | *11 Below* | 0.0008 |
| *5 Above* | 0.0012 | *11 Above* | 0.0005 |
| *6 Below* | 0.0015 | *12 Below* | 0.0006 |

# 8. TWELVE STOREY BUILDING "B"

According to the Loadings Standard (Standards New Zealand. 1992), the maximum allowable inter-storey drift when designed using a modal method of analysis is 1.50% for a total height of the frame of 40.8 m. For non-linear time-history analyses, the maximum inter-storey drift is 2.50%.

In the 12 storey B structure smaller beam and column section sizes were used in the upper 6 storeys compared with the corresponding values in the lower 6 storeys. For more details on the structure properties see section 5.12 and Appendix A.

## *8.1 Modal Analysis Results*

The dynamic properties of the 12 storey B frame are listed in Table 8.1. The square root sum of the squares combination method was used.

**Table 8-1 Dynamic properties of the 12 storey B frame**

| *Mode* | *Effective mass (Tonnes)* | *% Mass (Cumulative)* | *Period (seconds)* | *$V_{max}$ (kN)* |
|---|---|---|---|---|
| *1* | 2108 | 74 | 2.11 | ***682.4*** |
| *2* | 333.1 | 91 | 0.74 | ***281.4*** |
| *3* | 94.1 | 94 | 0.42 | ***140.3*** |
| *4* | 61.24 | 96 | 0.29 | ***91.3*** |
| *5* | 30.17 | 98 | 0.22 | ***45.0*** |
| *6* | 24 | 98 | 0.17 | ***35.8*** |
| *$\Sigma$=* | 2650.61 | | ***SRSS =*** | ***759.1**** |

***Results will be properly scaled to match the minimum base shear set by the NZS 1170.5 draft.***

As mentioned in section 6.2, the NZS 1170.5 draft sets a minimum design base shear of 3.2% of the total weight of the building, which would result in a base shear of 844.8 kN. Hence the modal values were scaled by 1.11 (*844.8 / 759.1 = 1.11)* to comply with this minimum base shear. Table 8.2 shows the modal combined inter-storey drifts, P-delta forces factorised by β and the total inter-storey drifts, which is the addition of combined modal and P-delta inter-storey drifts.

**Table 8-2 Combined modal inter-storey drifts, P-delta induced forces and total inter-storey drifts of structure 12B**

| *Storey* | *Combined modal inter-storey drifts (%)* | *P-delta forces* (kN)* | *Combined modal plus P-delta inter-storey drifts (%)* |
|---|---|---|---|
| *1* | 0.77 | -73.87 | 1.16 |
| *2* | 1.00 | 52.52 | 1.55 |
| *3* | 0.97 | 64.70 | 1.51 |
| *4* | 0.92 | 60.66 | 1.38 |
| *5* | 0.85 | 52.93 | 1.25 |
| *6* | 0.80 | -5.21 | 1.14 |
| *7* | 0.95 | 33.10 | 1.38 |
| *8* | 0.99 | 62.34 | 1.42 |
| *9* | 0.87 | 57.55 | 1.20 |
| *10* | 0.71 | 45.79 | 0.93 |
| *11* | 0.53 | 31.25 | 0.66 |
| *12* | 0.33 | 13.95 | 0.39 |
| | $\Sigma =$ | *395.73* | |

**Note that P-delta forces have already been factorised by $\beta$.*

### *8.2 Non-linear Time-history Analyses Results*

Figures 8.1 and 8.2 show the top floor displacement versus time and maximum inter-storey drifts with and without P-delta effects for model 12B/6/E/T when subject to ground acceleration records Modified El Centro EW and Modified TAFT 1. As seen for the modified ground acceleration TAFT 1, at the second storey, considering P-delta effects could be the difference between exceeding the 2.5% inter-storey drift limit or not. P-delta effects have a greater effect to the maximum inter-storey drift of the lower levels and a major influence on residual displacements.

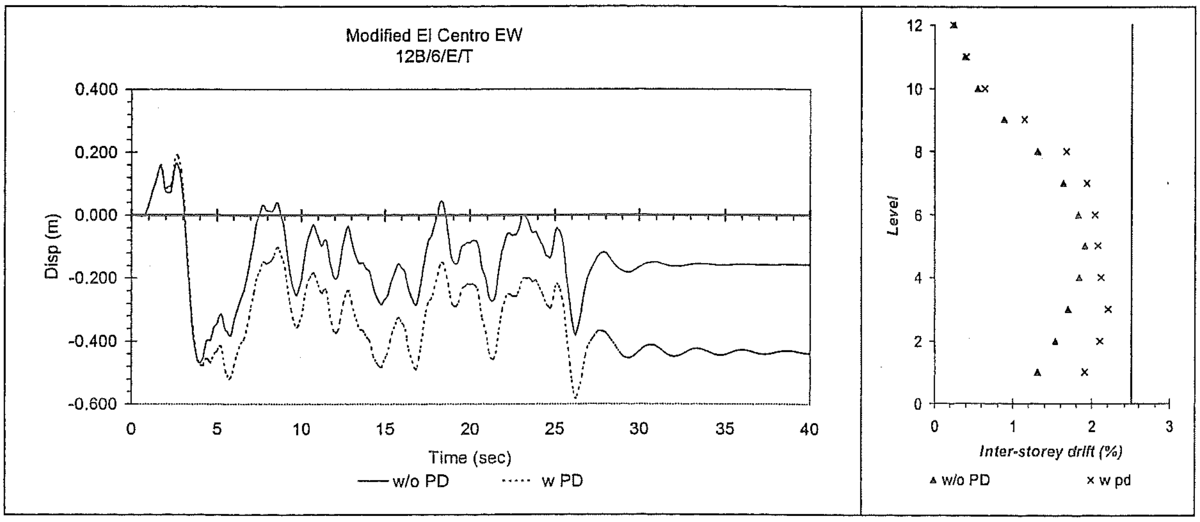



**Figure 8-1 Top floor displacement versus time and maximum inter-storey drifts for model 12B/6/E/T for modified ground acceleration record El Centro EW**

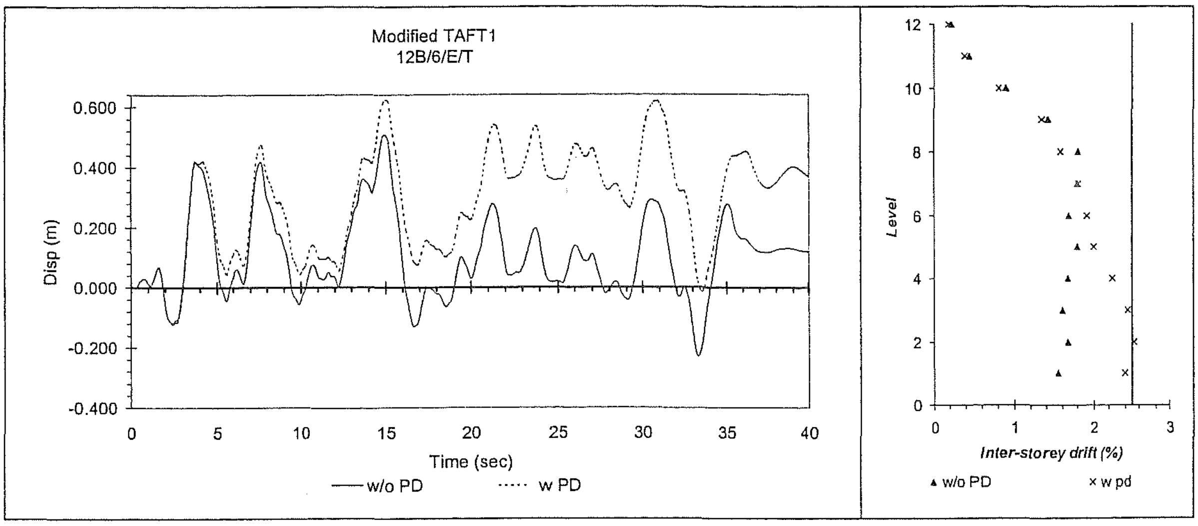



**Figure 8-2 Top floor displacement versus time and maximum inter-storey drifts for model 12B/6/E/T for modified ground acceleration records TAFT 1**

### *8.2.1 Axial load variation in non-linear time-history analyses*

Because the rotations of interior columns will be studied, analyses were made to understand their axial load variation. Table 8.3 shows the design axial load for the interior columns of model 12B/6/3101/T and a ratio between the design axial load over the maximum and the minimum axial load from the non-linear time-history analyses using the four modified ground acceleration records.

**Table 8-3 Ratio between design axial load in columns over maximum and minimum column axial load values from non-linear time-history analyses to structure 12B/6/3101/T**

| *Column design axial load over maximum and minimum axial load from non-linear time-history analyses* | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| *Level* | *Design (kN)* | *El Centro NS* | | *EL Centro EW* | | *TAFT 1* | | *TAFT 2* | | |
| | | *Max* | *Min* | *Max* | *Min* | *Max* | *Min* | *Max* | *Min* |
| 1 | -1401.9 | 1.06 | 0.95 | 1.06 | 0.96 | 1.06 | 0.95 | **1.07** | 0.96 |
| 2 | -1276.5 | 1.05 | 0.97 | 1.05 | 0.97 | 1.05 | 0.96 | **1.07** | 0.97 |
| 3 | -1151.1 | 1.04 | 0.96 | **1.06** | 0.98 | 1.05 | 0.96 | **1.06** | 0.97 |
| 4 | -1025.6 | 1.04 | 0.96 | **1.06** | 0.98 | 1.05 | 0.97 | **1.06** | 0.97 |
| 5 | -900.2 | 1.05 | 0.98 | **1.07** | 0.98 | 1.06 | 0.97 | **1.07** | 0.97 |
| 6 | -774.8 | 1.06 | 0.97 | **1.07** | 0.98 | **1.07** | 0.97 | **1.07** | 0.95 |
| 7 | -649.4 | 1.07 | 0.97 | 1.08 | 0.98 | 1.07 | 0.97 | **1.09** | 0.97 |
| 8 | -538.8 | 1.08 | 0.96 | **1.10** | 0.97 | 1.08 | 0.96 | **1.10** | 0.97 |
| 9 | -428.2 | 1.09 | 0.96 | **1.12** | 0.96 | 1.10 | 0.96 | **1.12** | 0.97 |
| 10 | -317.6 | 1.12 | 0.95 | **1.13** | 0.94 | 1.12 | 0.96 | **1.13** | 0.96 |
| 11 | -207.0 | 1.16 | 0.94 | 1.15 | 0.92 | 1.16 | 0.96 | **1.17** | 0.94 |
| 12 | -96.4 | 1.18 | 0.93 | 1.15 | 0.92 | **1.22** | 0.95 | 1.17 | 0.93 |

Table 8.4 shows the design reinforcement ratio ($\rho_{design}$) of the interior columns and the reinforcement ratio if the columns would have been designed using the most critical axial load from the non-linear time-history analyses. The variation of these values is negligible for all practical purposes.

**Table 8-4 Interior column reinforcement ratio variation**

| *Level* | $\rho_{design}$ | $\rho_{max}$ | $\rho_{max} / \rho_{design}$ |
|---|---|---|---|
| 1 | 0.0099 | 0.0103 | 1.04 |
| 2 | 0.0121 | 0.0125 | 1.03 |
| 3 | 0.0141 | 0.0145 | 1.03 |
| 4 | 0.0145 | 0.0149 | 1.03 |
| 5 | 0.0133 | 0.0137 | 1.03 |
| 6 | *0.0167 / 0.0185** | 0.0169 / 0.0187 * | 1.01 / 1.01 * |
| 7 | 0.0143 | 0.0147 | 1.03 |
| 8 | 0.0149 | 0.0152 | 1.02 |
| 9 | 0.0167 | 0.0171 | 1.02 |
| 10 | 0.0189 | 0.0191 | 1.01 |
| 11 | 0.0145 | 0.0147 | 1.01 |
| 12 | 0.0022 | 0.0022 | 1.00 |

** The first value corresponds to the reinforcement ratio below the beam and the second to that above the beam. Since the column cross section changes at this level, even for continuous steel the reinforcement ratio changes.*

### *8.2.2 Comparison between frames with different column characteristic*

A summary of the results from models 12B/6/E/T and 12B/6/3101/T analysed using the 8 earthquake records described in section 5.2.2, are presented. The main characteristics of these structures are stated below. For more details see section 5.12 and Appendix A.

1. Structure 12B/6/E/T
   - Designed to a ductility of 6
   - Levels on the bottom half have different beam and column cross-sections as those in the upper half
   - All beams comply with minimum steel requirements
   - Columns are elastic except at the base
   - A Takeda hysteretic model is used for all elements

2. Structure 12B/6/3101/T
   - Designed to a ductility of 6
   - Levels on the bottom half have different beam and column cross-sections as those in the upper half

- All beams comply with minimum steel requirements
- Columns are designed to NZS 3101:1995 (Standards New Zealand. 1995)
- A Takeda hysteretic model is used for all elements

Appendix B, part two, shows the response of structures 12B/6/E/T and 12B/6/3101/T when subject to each of the selected ground acceleration records. The same results as those described for the six storey structures are reported.

The non-linear time-history analyses to structure 12B/6/1170/T could not be run. The structural analysis program (Carr 2003) gave an error message *"Not a number in residual displacements"* which is probably caused by an underflow. The program is being adjusted to avoid this problem. Due to time limitations, the results from these analyses are not included in this work. The same problem arose when non-linear time-history analyses were run for the 2,500 years return period earthquakes for structures 12B/6/3101/T and 12B/6/1170/T.

### *8.2.2.1 Base shear*

Table 8.5 shows the average maximum base shear and standard deviation of structures 12B/6/E/T and 12B/6/3101/T for the non-linear time-history analyses. It also gives the equivalent static and combined modal base shear and the P-delta forces used for each of these analyses. The modal base shear has been scaled to meet the minimum requirements of the draft 1170.5.

**Table 8-5 Base shear summary for structures 12B/6/E/T and 12B/6/3101/T**

| | Eq. Static (kN) | Modal (kN) | | 12B/6/E/T (kN) | | 12B/6/3101/T (kN) | |
|---|---|---|---|---|---|---|---|
| | | | | Average | Std. Dev. | Average | Std. Dev. |
| *Base shear* | 865.0 | 844.8 | *Without P-Δ* | 1771.0 | 59.7 | 1794.9 | 56.6 |
| *P-Δ induced** | 351.6 | 395.7 | *With P-Δ* | 2031.8 | 179.2 | 2023.7 | 172.9 |
| *Σ=* | 1216.6 | 1240.5 | | | | | |

**P-delta forces are already multiplied by the factor β.*

The difference in the averaged maximum base shears between structures 12B/6/E/T and 12B/6/3101/T is less than 1% for both cases, with and without P-delta effects. The averaged base shear with P-delta effects for the non-linear time-history analyses is equivalent to 7.6% of the total weight of the structure which shows an increase of two fold with the minimum design base shear. The averaged maximum base shear for the non-linear time-history analyses that used the modified ground acceleration records when P-delta effects are considered, have a 6% increase compared to the averaged maximum base shear when using the natural ground acceleration records. Structure 12B/6/3101/T shows a 5% increase in the averaged base shear from the non-linear time-history analyses when compared to the corresponding value of structure 12A/6/3101/T. The averaged base shear from the non-linear time-history analyses of structures 12B/6/3101/T and 12B/6/E/T exceeds the modal plus P-delta base shear by 60%. Appendix C shows how the damping forces affect the base shear.

#### *8.2.2.2 Inter-storey drifts*

Table 8.6 shows the averaged maximum inter-storey drifts and the corresponding standard deviations for the non-linear time-history analyses of structures 12B/6/E/T and 12B/6/3101/T with and without P-delta effects. It also shows the ratio of the average values of maximum inter-storey drifts obtained when P-delta effects are considered to the corresponding values without P-delta effects. None of the averaged values for the averaged maximum inter-storey drifts is greater than the 2.5% allowed for the non-linear time-history analyses. Furthermore, the averaged maximum inter-storey drifts from the non-linear time-history analyses of both structures is equal or less to the 1.5% inter-storey drift limit allowed for elastic analyses. An increase of close to 30% is observed in both structures when the averaged maximum inter-storey drifts considering P-delta are compared to the results when P-delta effects are omitted. It is interesting to see that when the averaged maximum inter-storey drifts of structure 12B/6/3101/T are compared to those of structure 12A/6/3101/T, levels 1 to 4 a reduction of up to 12% of the inter-storey drift for structure 12B/6/3101/T. From levels 5 upwards, structure 12B/6/3101/T shows an increase in its averaged maximum inter-storey drifts g between 34% and 143% of the inter-storey drift at storey 7.

**Table 8-6 Average maximum inter-storey drifts and standard deviations considering P-delta effects using eight ground acceleration records for structures 12B/6/E/T and 12B/6/3101/T**

| | *12B/6/E/T* | | | *12B/6/3101/T* | | |
|---|---|---|---|---|---|---|
| ***Storey*** | ***Avg.*** *(%)* | ***Std. dev.*** *(%)* | ***I. Drift WPD / W/o PD*** | ***Avg.*** *(%)* | ***Std. dev.*** *(%)* | ***I. Drift WPD / W/o PD*** |
| *1* | 1.36 | 0.53 | 1.30 | 1.30 | 0.52 | 1.28 |
| *2* | 1.50 | 0.53 | 1.25 | 1.49 | 0.53 | 1.24 |
| *3* | 1.50 | 0.55 | 1.18 | 1.50 | 0.54 | 1.17 |
| *4* | 1.47 | 0.52 | 1.12 | 1.48 | 0.53 | 1.12 |
| *5* | 1.46 | 0.49 | 1.08 | 1.47 | 0.49 | 1.08 |
| *6* | 1.41 | 0.47 | 1.07 | 1.42 | 0.48 | 1.07 |
| *7* | 1.39 | 0.36 | 1.02 | 1.39 | 0.38 | 1.03 |
| *8* | 1.29 | 0.28 | 0.96 | 1.29 | 0.28 | 0.96 |
| *9* | 1.02 | 0.24 | 0.95 | 1.02 | 0.23 | 0.95 |
| *10* | 0.65 | 0.15 | 0.93 | 0.65 | 0.14 | 0.91 |
| *11* | 0.37 | 0.05 | 0.92 | 0.38 | 0.06 | 0.88 |
| *12* | 0.20 | 0.03 | 0.90 | 0.30 | 0.06 | 0.87 |

Figure 8.3 shows the maximum inter-storey drifts for the non-linear time-history analyses of structure 12B/6/3101/T. None of the values exceed the 2.5% allowable limit. The averaged inter-storey drift from the non-linear time-history analyses using the modified ground acceleration records compared to the corresponding value when using the natural records shows an increase of up to 77% at level 4 when P-delta effects are considered.

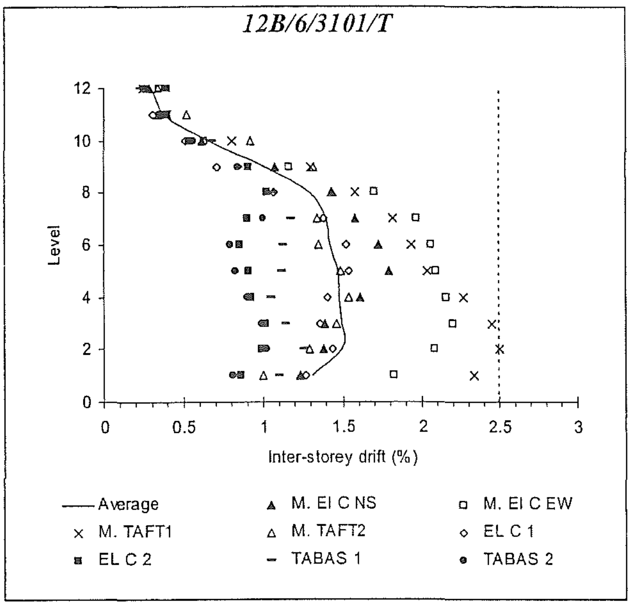



**Figure 8-3 Maximum inter-storey drifts for the non-linear time-history analyses of structure 12B/6/3101/T with P-delta effects**

### *8.2.2.3 Maximum displacements*

Table 8.7 shows the averaged displacements and standard deviations from the non-linear time-history analyses of structures 12B/6/E/T and 12B/6/3101/T considering P-delta effects. There is a negligible difference between the averaged maximum displacements of structure 12B/6/E/T and 12B/6/3101/T. The averaged maximum displacements when P-delta is considered vary from the corresponding values without considering P-delta in the same 30% as for the averaged maximum inter-storey drifts.

**Table 8-7 Averaged maximum displacements and standard deviations for non-linear time-history analyses of structures 12B/6/E/T and 12B/6/3101/T considering P-delta effects**

| Level | *12B/6/E/T* | | *12B/6/3101/T* | |
|---|---|---|---|---|
| | Average (m) | Std Deviation (m) | Average (m) | Std Deviation (m) |
| 1 | 0.046 | 0.017 | 0.044 | 0.017 |
| 2 | 0.098 | 0.034 | 0.095 | 0.033 |
| 3 | 0.147 | 0.051 | 0.144 | 0.051 |
| 4 | 0.192 | 0.069 | 0.190 | 0.069 |
| 5 | 0.233 | 0.086 | 0.231 | 0.086 |
| 6 | 0.271 | 0.101 | 0.271 | 0.101 |
| 7 | 0.313 | 0.114 | 0.311 | 0.113 |
| 8 | 0.350 | 0.121 | 0.348 | 0.120 |
| 9 | 0.376 | 0.125 | 0.375 | 0.125 |
| 10 | 0.392 | 0.128 | 0.391 | 0.127 |
| 11 | 0.400 | 0.128 | 0.399 | 0.127 |
| 12 | 0.404 | 0.128 | 0.405 | 0.126 |

Figure 8.4 shows the maximum displacements from the non-linear time-history analyses of structure 12B/6/3101/T. The maximum displacements when using the records Modified El Centro EW and Modified TAFT 1 records appear to be the critical cases. It is only with these two ground motion records that the lateral displacements exceed the values predicted by the modal analysis. The averaged maximum displacements from the non-linear time-history analyses match the design displaced shape within a 5% margin between levels 2 and 8 and show a 13% greater displacement at level 1.

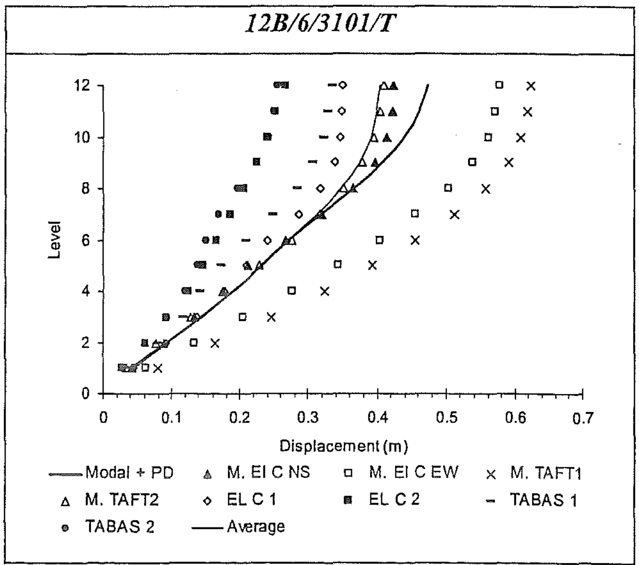



**Figure 8-4 Maximum displacements from non-linear time-history analyses of structure 12B/6/3101/T considering P-delta effects**

### *8.2.2.4 Beam rotations*

The beam rotations from the non-linear time-history analyses and the demand beam rotations from the equivalent static and modal analyses were calculated as described in section 6.3.4.4. Figure 8.5 shows the averaged maximum beam rotations from the non-linear time-history analyses for structures 12B/6/E/T and 12B/6/3101/T. It also shows the design total rotations from the equivalent static and modal analyses. Table 8.8 gives the averaged beam rotation values from figure 8.5 and its standard deviation together with the beam rotation demand calculated from the modal and static analyses. The demand in beam rotations from the modal analysis is greater at levels 1 to 4 to that from the equivalent static analysis. This is because for the modal analysis the design base shear had to be scaled so that it would match the minimum allowable design base shear and hence, affected the inter-storey drifts[1]. The averaged maximum beam rotations show no difference between the two structures. All the average beam rotation values are considerably below the beam rotations from the modal and static analyses.

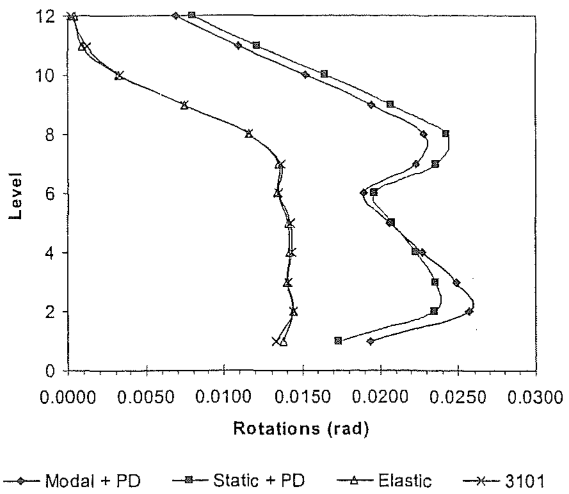



**Figure 8-5 Equivalent static and modal analysis beam rotation demand and average of maximum beam rotations from non-linear time-history analyses of structures 12B/6/E/T and 12B/6/3101/T**

[1] Theoretically, the structural ductility factor could have been adjusted to match the minimum base shear and this would have reduced the displacements given by the equivalent static and modal methods.

**Table 8-8 Equivalent static and modal analysis beam rotation demand and average of maximum beam rotations and standard deviations from non-linear time-history analyses of structures 12B/6/E/T and 12B/6/3101/T**

| Level | Eq. static rotations* (rad) | Modal rotations* (rad) | 12B/6/E/T Average rot. (rad) | 12B/6/E/T Std. Dev. (rad) | 12B/6/3101/T Average rot. (rad) | 12B/6/3101/T Std. Dev. (rad) |
|---|---|---|---|---|---|---|
| *1* | 0.0173 | 0.0193 | 0.0138 | 0.0058 | 0.0133 | 0.0059 |
| *2* | 0.0235 | 0.0257 | 0.0145 | 0.0060 | 0.0144 | 0.0059 |
| *3* | 0.0236 | 0.0249 | 0.0140 | 0.0061 | 0.0142 | 0.0061 |
| *4* | 0.0223 | 0.0228 | 0.0142 | 0.0056 | 0.0144 | 0.0056 |
| *5* | 0.0208 | 0.0206 | 0.0141 | 0.0054 | 0.0143 | 0.0055 |
| *6* | 0.0196 | 0.0189 | 0.0133 | 0.0047 | 0.0134 | 0.0049 |
| *7* | 0.0236 | 0.0223 | 0.0134 | 0.0027 | 0.0136 | 0.0027 |
| *8* | 0.0243 | 0.0228 | 0.0116 | 0.0023 | 0.0116 | 0.0020 |
| *9* | 0.0207 | 0.0194 | 0.0074 | 0.0017 | 0.0074 | 0.0017 |
| *10* | 0.0164 | 0.0152 | 0.0032 | 0.0008 | 0.0033 | 0.0010 |
| *11* | 0.0121 | 0.0109 | 0.0009 | 0.0004 | 0.0012 | 0.0006 |
| *12* | 0.0079 | 0.0068 | 0.0003 | 0.0001 | 0.0002 | 0.0000 |

Figure 8.6 shows the scatter of the maximum rotations from the non-linear time-history analyses of structure 12B/6/3101/T. The maximum beam rotations when using ground acceleration records Modified El Centro EW and Modified TAFT 1 match closely on the first 6 levels the beam rotations from the modal analysis.

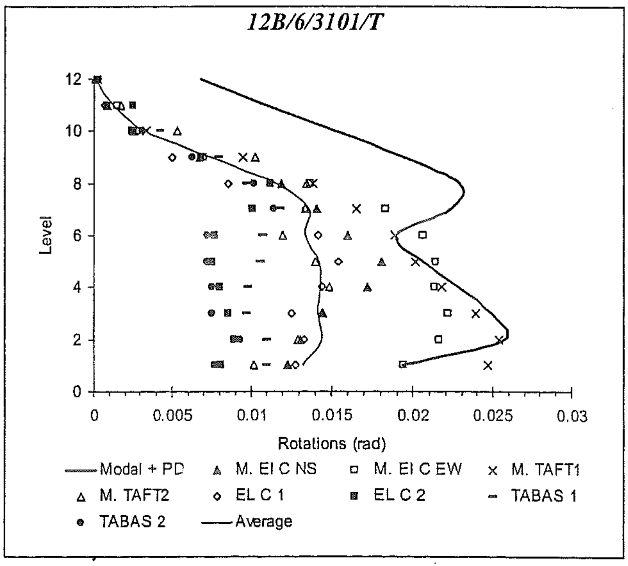



**Figure 8-6 Maximum beam rotations of structure 12B/6/3101/T from non-linear time-history analyses**

Table 8.9 shows the averaged maximum beam section ductilities for structures 12B/6/E/T and 12B/6/3101/T from the non-linear time-history analyses and the beam ductility demand from the equivalent static and modal analyses. For both structures the highest average section ductility is 24 which is well below the rotation ductility of 38 and 40 that result from the equivalent and modal analyses.

**Table 8-9 Design beam section ductilities and averaged maximum beam section ductilities for structures 12B/6/E/T and 12B/6/3101/T**

| Level | Eq *static* | Modal | 12B/6/E/T | 12B/6/3101/T |
|---|---|---|---|---|
| | $S_\mu$ | $S_\mu$ | $S_\mu$ | $S_\mu$ |
| 1 | 24.1 | 26.9 | 19.1 | 18.4 |
| 2 | 30.9 | 33.8 | 19.1 | 19.0 |
| 3 | 33.4 | 35.3 | 19.8 | 20.1 |
| 4 | 34.9 | 35.6 | 22.2 | 22.5 |
| 5 | 35.8 | 35.5 | 24.3 | 24.6 |
| 6 | 33.8 | 32.6 | 22.9 | 23.1 |
| 7 | 35.8 | 34.0 | 20.5 | 20.7 |
| 8 | 40.6 | 38.2 | 19.3 | 19.4 |
| 9 | 34.7 | 32.5 | 12.4 | 12.5 |
| 10 | 27.5 | 25.4 | 5.3 | 5.5 |
| 11 | 20.3 | 18.3 | 1.4 | 2.1 |
| 12 | 13.2 | 11.4 | 0.5 | 0.3 |

### *8.2.2.5 Interior column rotations*

Table 8.10 shows the yield rotation, the averaged maximum rotation for the non-linear time-history analyses, its standard deviation and the averaged maximum section curvature ductility for an interior column of structure 12B/6/3101/T. Besides the base of the column and the top floor column were plastic hinges are allowed and level 1, which shows very small inelastic behaviour, all the other levels show no inelastic behaviour.

**Table 8-10 Interior columns design yield rotation, averaged maximum rotation and standard deviation for non-linear time-history analyses of structure 12B/6/3101/T**

| | *12B/6/3101/T* | | | | | *12B/6/3101/T* | | | |
|---|---|---|---|---|---|---|---|---|---|
| *level* | $\theta_y$ *(rad)* | *Avg. rot (rad)* | *Std. dev. (rad)* | $S_\mu$ | *level* | $\theta_y$ *(rad)* | *Avg. rot (rad)* | *Std. dev. (rad)* | $S_\mu$ |
| *G.L.* | 0.0013 | 0.0094 | 0.0044 | 7.0 | *6 Above* | 0.0013 | 0.0012 | 0.0002 | 0.9 |
| *1 Below* | 0.0014 | 0.0007 | 0.0000 | 0.5 | *7 Below* | 0.0012 | 0.0011 | 0.0002 | 0.9 |
| *1 Above* | 0.0014 | 0.0014 | 0.0004 | 1.1 | *7 Above* | 0.0012 | 0.0010 | 0.0001 | 0.8 |
| *2 Below* | 0.0013 | 0.0012 | 0.0001 | 0.9 | *8 Below* | 0.0013 | 0.0012 | 0.0001 | 0.9 |
| *2 Above* | 0.0013 | 0.0011 | 0.0002 | 0.8 | *8 Above* | 0.0013 | 0.0008 | 0.0001 | 0.6 |
| *3 Below* | 0.0013 | 0.0011 | 0.0001 | 0.8 | *9 Below* | 0.0013 | 0.0012 | 0.0001 | 1.0 |
| *3 Above* | 0.0013 | 0.0010 | 0.0001 | 0.8 | *9 Above* | 0.0013 | 0.0007 | 0.0001 | 0.5 |
| *4 Below* | 0.0014 | 0.0011 | 0.0001 | 0.8 | *10 Below* | 0.0012 | 0.0011 | 0.0001 | 0.9 |
| *4 Above* | 0.0014 | 0.0009 | 0.0001 | 0.7 | *10 Above* | 0.0012 | 0.0006 | 0.0001 | 0.5 |
| *5 Below* | 0.0013 | 0.0010 | 0.0001 | 0.8 | *11 Below* | 0.0012 | 0.0009 | 0.0001 | 0.7 |
| *5 Above* | 0.0013 | 0.0008 | 0.0001 | 0.6 | *11 Above* | 0.0012 | 0.0006 | 0.0001 | 0.5 |
| *6 Below* | 0.0013 | 0.0010 | 0.0001 | 0.8 | *12 Below* | 0.0010 | 0.0033 | 0.0007 | 3.3 |

Figure 8.7 shows the maximum rotations above and below the beam for structure 12B/6/3101/T. The maximum column rotation ductility at the base of the column is 13 for the ground acceleration record Modified Taft 1. At the top floor the maximum column rotation ductility is 4.5 for the ground acceleration record El Centro 1. From levels 1 through 11 the maximum column rotation ductility is 1.7 at level 1 above the beam for the modified ground acceleration El Centro NS.

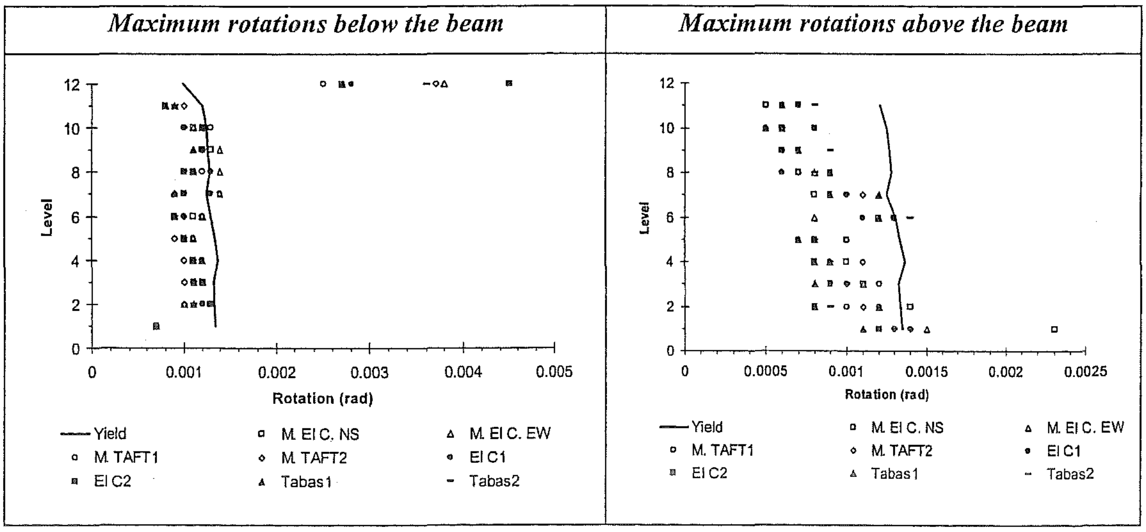



**Figure 8-7 Column maximum rotations above and below the beam for non-linear time-history analyses of structure 12B/6/3101/T**

# 9. FOUR STOREY BUILDING

The maximum allowable inter-storey drift according to the Loadings Standard 4203:1992 (Standards New Zealand. 1992) when designed using the equivalent static or modal methods of analysis for a total height of the frame of 13.6 m, is 2.00%. For non-linear time-history analyses, the maximum allowable inter-storey drift is 2.50%. For the details of the structure see section 5.12 and appendix A.

## *9.1 Equivalent Static Analysis Results*

The fundamental period estimated by the Rayleigh method was 1.10 seconds. The calculated base shear for the equivalent static analysis is 557.2 kN and the calculated maximum inter-storey drift for the equivalent static analysis after adding P-delta effects was 1.99 % of the storey height. Columns two and three in Table 9.1 show the resulting forces and inter-storey drifts from the equivalent static analysis. Column four shows the P-delta induced forces factorised by the $\beta$ factor and column five the total inter-storey drifts. The inter-storey drifts were reduced, as detailed in clause 4.8.1.5 of the Loadings Standard, NZS 4203:1992 (Standards New Zealand. 1992) and then scaled by the ductility factor to account for inelastic deformations.

P-delta effects have a significant influence to the design of this four storey structure when in practice the P-delta effects could easily be overlooked. For this case, forces of the order of 35% of the base shear from the equivalent static analysis are added in order to give the structure the appropriate strength to sustain P-delta effects.

**Table 9-1 Equivalent static analysis results for the four storey frame**

| *Storey* | *Eq static Forces (kN)* | *Eq. Static Inter-storey drifts (%)* | *P-delta Forces* (kN)* | *Eq static plus P-delta Inter-storey drifts (%)* |
|---|---|---|---|---|
| *1* | 51.26 | 1.12 | 10.25 | 1.58 |
| *2* | 102.53 | 1.41 | 84.09 | 1.99 |
| *3* | 153.79 | 1.16 | 70.25 | 1.53 |
| *4* | 249.64 | 0.73 | 32.00 | 0.88 |
| *Σ=* | *557.22* | | *196.58* | |

**Note that P-delta forces have been factorised by $\beta$.*

## *9.2 Modal Analysis Results*

Table 9.2 shows the dynamic properties of the four storey frame. The SRSS combination method was used. Table 9.3 shows the combined modal inter-storey drifts, the P-delta forces and the total inter-storey drifts. These drifts have been scaled for inelastic action as required in NZS 4203:1992

(Standards New Zealand. 1992). For this structure the minimum base shear requirements are not critical. The combined modal base shear is of the order of 5.8 % of the total weight.

**Table 9-2 Dynamic properties of the four storey frame**

| *Mode* | *Effective mass (tonnes)* | *% Mass (Cumulative)* | *Period (seconds)* | $S_{ai}$ *(%g)* | $V_{max}$ *(kN)* |
|---|---|---|---|---|---|
| *1* | 766 | *85* | 1.10 | 0.063 | 474.9 |
| *2* | 92.13 | *96* | 0.34 | 0.152 | 137.4 |
| *3* | 30.78 | *99* | 0.19 | 0.152 | 45.9 |
| *4* | 81.64 | *100* | 0.13 | 0.152 | 121.7 |
| Σ = | 970.55 | | | *SRSS* = | 511.2 |

**Table 9-3 Modal inter-storey drifts and P-delta forces for four storey structure**

| *Storey* | *Modal combined Inter-storey drifts (%)* | *P-delta Forces (kN)* | *Combined modal plus P-delta Inter-storey drifts (%)* |
|---|---|---|---|
| *1* | 1.15 | 18.94 | 1.62 |
| *2* | 1.39 | 87.97 | 1.95 |
| *3* | 1.08 | 66.86 | 1.43 |
| *4* | 0.64 | 28.34 | 0.78 |
| | *Σ=* | *202.11* | |

**Note that P-delta forces have been factorised by β.*

## *9.3 Non-linear Time-history Analyses Results*

Figures 9.1 and 9.2 show the top floor displacement versus time when structure 4/6/E/T is subject to modified ground acceleration records El Centro EW and TAFT 1 respectively.

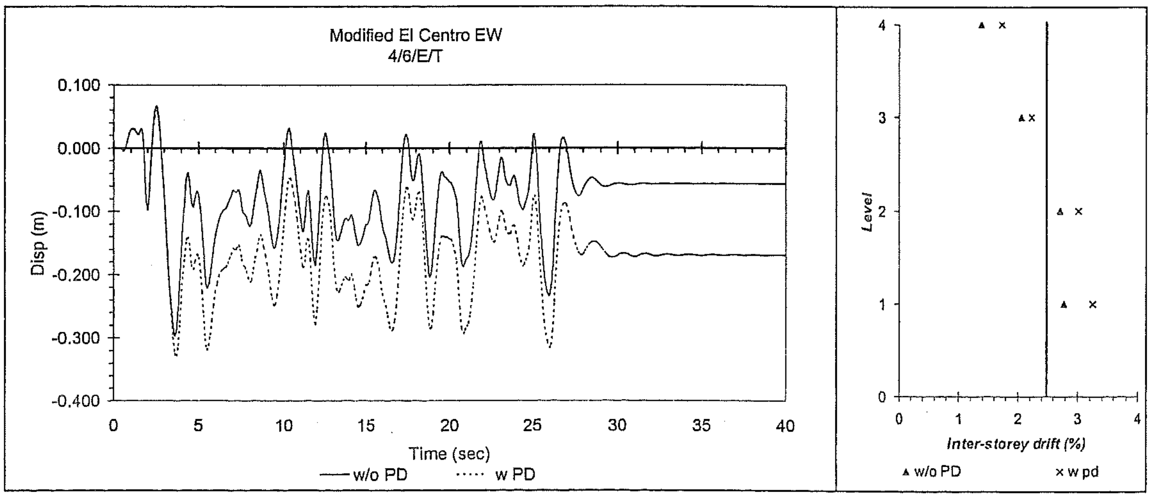



**Figure 9-1 Model 4/6/E/T top floor displacement versus time and maximum inter-storey drifts response with and without P-delta for modified ground acceleration record El Centro EW**

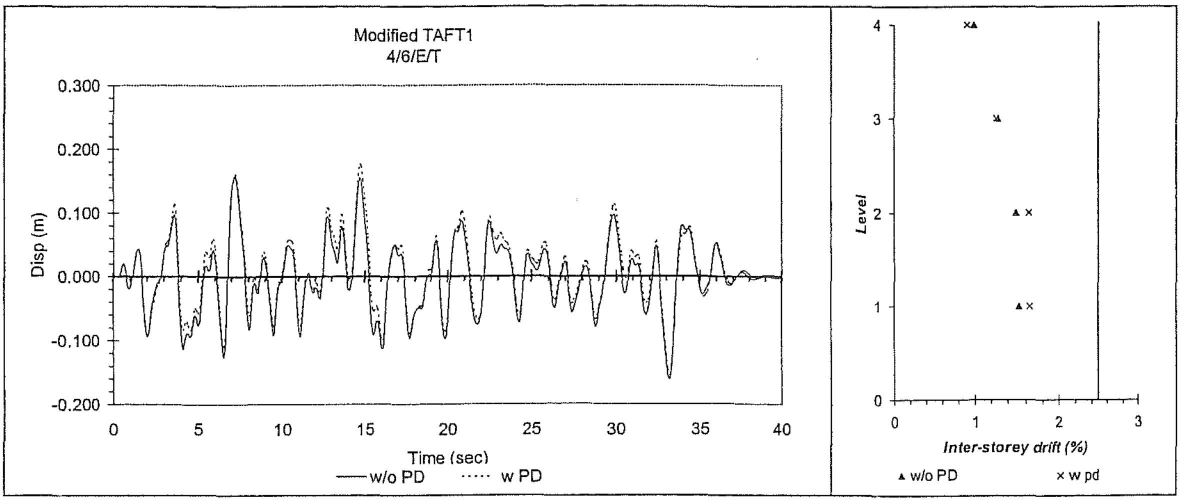



**Figure 9-2 Model 4/6/E/T top floor displacement versus time and maximum inter-storey drifts response with and without P-delta for modified ground acceleration record TAFT 1**

P-delta actions had a major influence on the response of structure 4/6/E/T when it was subjected to the modified El Centro EW ground acceleration record but very little influence when the analysis was made using the Taft 1 record. It should be noted that the Modified El Centro EW record was the most critical of the eight records used in the non-linear time-history analyses for this structure.

Figures 9.3 and 9.4 show the top floor displacement history and the maximum inter-storey drifts for structures 4/6/3101/T and 4/6/1170/T when using the modified ground acceleration record El Centro EW. The maximum inter-storey drift at storey one from structure 4/6/E/T of 3.26 % is slightly greater than the corresponding value for structure 4/6/3101/T of 3.19%. Structure 4/6/1170/T shows the greatest inter-storey drift of 4.5% at storeys one and two when using the modified El Centro EW ground acceleration record. An inter-storey displacement as high as 4.5%, could compromise the stability of the structure. Structures 4/6/3101/T and 4/6/1170/T show an increase of 1.21 and 1.59 times its maximum inter-storey drifts when P-delta effects are included compared to the corresponding values when they are not.

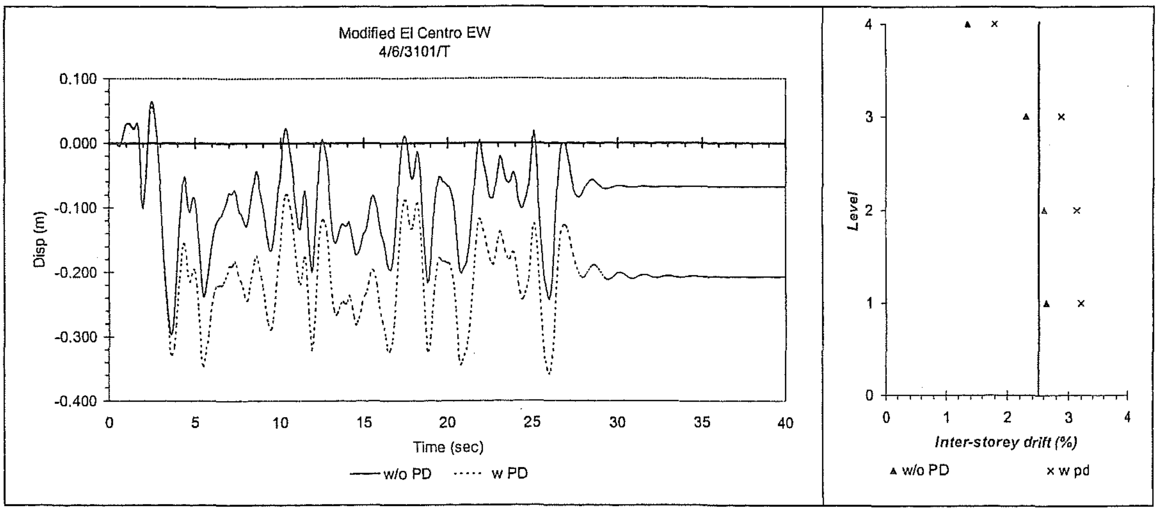



**Figure 9-3 Model 4/6/3101/T top floor displacement versus time and maximum inter-storey drifts response with and without P-delta for modified ground acceleration record El Centro EW**

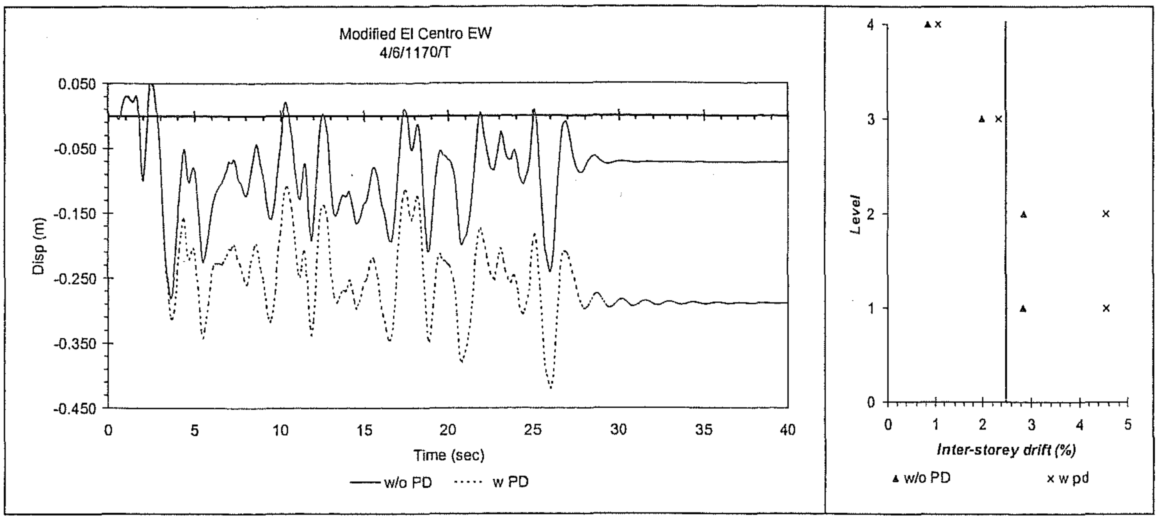



**Figure 9-4 Model 4/6/1170/T top floor displacement versus time and maximum inter-storey drifts response with and without P-delta for modified ground acceleration record El Centro EW**

### *9.3.1 Axial load variation*

As mentioned before, just the variation of axial load in the interior columns is analysed because their rotation is studied later. Table 9.4 shows the design axial load for the interior columns of model 4/6/3101/T and a ratio between the design axial load over the maximum and the minimum axial load obtained from the non-linear time-history analyses using the four modified ground acceleration records. Table 9.5 shows the design reinforcement ratio ($\rho_{design}$) of the interior columns and the reinforcement ratio if these columns would have been designed using the most critical axial load resulting from the non-linear time-history analyses. The variation of the steel reinforcement when the columns were designed with the maximum axial load from the non-linear time-history analyses is negligible for all practical purposes.

**Table 9-4 Ratio between the design axial load of interior columns over maximum and minimum axial load values for non-linear time-history analyses of structure 4/6/3101/T**

| *Column design axial load over maximum and minimum axial load from non-linear time-history analyses* | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| *Level* | *Design values (kN)* | *El Centro NS* | | *EL Centro EW* | | *TAFT 1* | | *TAFT 2* | | |
| | | *Max* | *Min* | *Max* | *Min* | *Max* | *Min* | *Max* | *Min* |
| 1 | -311.3 | **1.10** | 0.94 | 1.07 | 0.92 | 1.07 | 0.92 | 1.09 | 0.93 |
| 2 | -231.0 | 1.08 | 0.95 | 1.07 | 0.93 | 1.08 | 0.94 | **1.12** | 0.93 |
| 3 | -150.7 | 1.08 | 0.94 | 1.09 | 0.90 | 1.09 | 0.92 | 1.17 | 0.93 |
| 4 | -70.4 | 1.15 | 0.88 | **1.20** | 0.90 | 1.19 | 0.85 | 1.18 | 0.88 |

**Table 9-5 Interior column reinforcement ratio variation for most critical axial load case from non-linear time-history analyses of structure 4/6/3101/T**

| *Level* | $\rho_{design}$ | $\rho_{max}$ | $\rho_{max} / \rho_{design}$ |
|---|---|---|---|
| 1 | 0.0176 | 0.0176 | 1.00 |
| 2 | 0.0326 | 0.0330 | 1.01 |
| 3 | 0.0115 | 0.0118 | 1.03 |
| 4 | 0.0054 | 0.0054 | 1.00 |

### *9.3.2 Comparison between Frames with Different Column Strengths*

In this section a summary of the results from the non-linear time-history analyses of models 4/6/E/T, 4/6/3101/T and 4/6/1170/T using 8 ground acceleration records is presented. The main characteristics of these models are listed below. For more detail see section 5.12 and appendix A.

1. Structure 4/6/E/T
   - Designed to a ductility of 6
   - All beams comply with minimum steel requirements
   - Beams at levels 3 and 4 have the same strength
   - Columns are elastic except at the base
   - A Takeda hysteretic model is used for all elements

2. Structure 4/6/3101/T
   - Designed to a ductility of 6
   - All beams comply with minimum steel requirements
   - Beams at levels 3 and 4 have the same strength
   - Columns are designed to NZS 3101:1995 (Standards New Zealand. 1995)
   - A Takeda hysteretic model is used for all elements

3. Structure 4/6/1170/T
   - Designed to a ductility of 6
   - All beams comply with minimum steel requirements
   - Beams at levels 3 and 4 have the same strength
   - Columns are designed to NZS 1170.5 draft for limited protection against plastic hinges
   - A Takeda hysteretic model is used for all elements

Appendix B, part two, shows the response of structures 4/6/E/T, 4/6/3101/T and 4/6/1170/T when subject to each of the selected ground acceleration records. The same results as those described for the six storey structure are reported.

*9.3.2.1 Base shear*

Table 9.6 shows the average peak base shear and the standard deviation for the non-linear time-history analyses using the 8 records without and with P-delta effects respectively for structures 4/6/E/T, 4/6/3101/T and 4/6/1170/T. It shows as well the Equivalent static and combined modal base shear values and the sum of the forces added in each analysis to account for P-delta effects.

For the equivalent static and modal analyses, the inclusion of P-delta effects represents an increase to the design forces of 35% and 39% of the design base shear respectively. For the non-linear time-history analyses, P-delta effects increase the averaged peak base shear by 16%, 11% and 14% for structures 4/6/E/T, 4/6/3101/T and 4/6/1170/T respectively compared to the corresponding value when P-delta effects are not included. Structure 4/6/E/T shows the greatest averaged peak base shear value when P-delta effects are considered by a 10% and 12% when compared to that of structures 4/6/3101/T and 4/6/1170/T. Structures 4/6/3101/T and 4/6/1170/T show a 2% difference in their averaged peak base shear when P-delta effects are considered compared to the corresponding values when they are not.

**Table 9-6 Base shear summary for structures 4/6/E/T, 4/6/3101/T and 4/6/1170/T**

| | **Eq. Static (kN)** | **Modal (kN)** |
|---|---|---|
| ***Base shear*** | 557.2 | 511.2 |
| ***P-Δ induced**** | 196.6 | 202.1 |
| ***Σ=*** | 753.8 | 713.3 |

| | **4/6/E/T (kN)** | | **4/6/3101/T (kN)** | | **4/6/1170/T (kN)** | |
|---|---|---|---|---|---|---|
| | **Average** | **Std. Dev.** | **Average** | **Std. Dev.** | **Average** | **Std. Dev.** |
| ***Without PD*** | 1064.2 | 88.8 | 1022.8 | 64.6 | 966.1 | 40.8 |
| ***With PD*** | 1243.5 | 148.9 | 1137.5 | 114.1 | 1107.6 | 161.7 |

**P-delta forces are already multiplied by the factor β.*

The averaged peak base shear with P-delta effects of structures 4/6/E/T, 4/6/3101/T and 4/6/1170/T is 1.74, 1.59 and 1.55 times greater respectively than the sum of the earthquake and P-delta design lateral forces from the modal analysis. Appendix C shows the importance of damping forces for the analytical models in the non-linear time-history analyses.

The averaged peak base shear from the non-linear analyses using the modified ground acceleration records is greater than the corresponding values when using the natural records by 6%, 4% and 9% for structures 4/6/E/T, 4/6/3101/T and 4/6/1170/T respectively.

*9.3.2.2 Inter-storey drifts*

Table 9.7 shows the average maximum inter-storey drifts for the eight ground acceleration records with P-delta effects for structures 4/6/E/T, 4/6/3101/T and 4/6/1170/T and its standard deviation. None of the averaged maximum inter-storey drifts exceed the allowable 2.5% limit. Structure 4/6/1170/T shows a large standard deviation of up to 1.21%. Structure 4/6/3101/T shows a reduction of its averaged maximum inter-storey drift of 28% at the first level when compared to its corresponding value for structure 4/6/E/T, but an increase at the third and fourth floors. Structure 4/6/1170/T shows a 29% increase in its averaged maximum inter-storey drift at the second storey when compared with that from structure 4/6/3101/T. The averaged maximum inter-storey drifts from the non-linear time-history analyses of structures 4/6/E/T and 4/6/3101/T do not exceed the 2.0% design limit set for elastic analyses.

**Table 9-7 Average maximum inter-storey drifts and standard deviations considering P-delta effects using eight ground acceleration records for structures 4/6/E/T, 4/6/3101/T and 4/6/1170/T**

| | | *4/6/E/T* | | *4/6/3101/T* | | *4/6/1170/T* | |
|---|---|---|---|---|---|---|---|
| | | *Avg.* | *Std. dev.* | *Avg.* | *Std. dev.* | *Avg.* | *Std. dev.* |
| | *Level* | (%) | (%) | (%) | (%) | (%) | (%) |
| *With P-delta* | *1* | 2.00 | 0.66 | 1.76 | 0.73 | 1.91 | 1.21 |
| | *2* | 1.90 | 0.59 | 1.85 | 0.63 | 2.38 | 1.02 |
| | *3* | 1.50 | 0.44 | 1.81 | 0.60 | 1.85 | 0.40 |
| | *4* | 1.07 | 0.44 | 1.43 | 0.47 | 0.95 | 0.20 |

Table 9.8 shows the ratio between the maximum average inter-storey drift with P-delta effects over the corresponding value without P-delta effects. Structure 4/6/3101/T and 4/6/1170/T show an increase of 12% and 26% in its averaged maximum inter-storey drifts when the non-linear time-history analyses consider P-delta effects compared to its corresponding value when P-delta effects are not considered.

**Table 9-8 Comparison between the maximum average inter-storey drifts with P-delta and without P-delta for structures 4/6/E/T, 4/6/3101/T and 4/6/1170/T**

| *Averaged maximum Inter-storey drifts With P-delta / Without P-delta* | | | |
|---|---|---|---|
| *Level* | *4/6/E/T* | *4/6/3101/T* | *4/6/1170/T* |
| *1* | 1.10 | 1.12 | 1.23 |
| *2* | 1.08 | 1.11 | 1.26 |
| *3* | 1.05 | 1.12 | 1.13 |
| *4* | 1.08 | 1.09 | 1.03 |

Figure 9.5 shows the maximum inter-storey drift from the non-linear time-history analyses of structures 46/E/T, 4/6/3101/T and 4/6/1170/T. For both structures, the Modified El Centro EW ground acceleration gives the most critical maximum inter-storey drifts. Structure 4/6/3101/T shows a maximum inter-storey drift at the first storey of 3.2% for the modified El Centro EW ground acceleration record. Structure 4/6/1170/T shows a maximum inter-storey drift for the modified ground acceleration record El Centro EW of 4.5% that would only avoid collapse if the structure was carefully

designed, detailed and built. This value has a great influence for the large standard deviation shown at storey 1 for this structure. The maximum inter-storey drifts at the first storey of structures 4/6/3101/T and 4/6/1170/T when using the modified El Centro EW ground acceleration record when P-delta effects are not considered are 2.6% and 2.8% respectively. For structure 4/6/3101/T, results show a similar scatter in all levels that is reflected in its standard deviation.

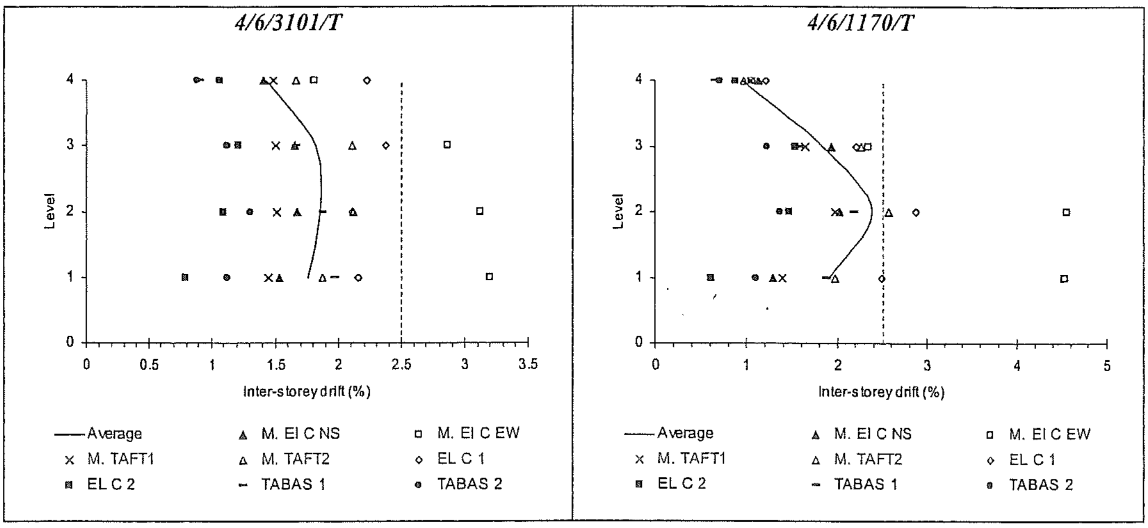



**Figure 9-5 Maximum inter-storey drifts and its average of structures 4/6/3101/T and 4/6/1170/T using eight ground acceleration records with P-delta effects**

Table 9.9 shows a comparison of the averaged maximum inter-storey drifts from the non-linear time history analyses using the four modified ground acceleration records and those resulting from the natural ground acceleration records. Structures 4/6/E/T and 4/6/3101/T show an increment in its averaged maximum inter-storey drifts of up to 30% and structure 4/6/1170/T of up to 50% when using the modified ground acceleration records compared to the averaged maximum inter-storey drifts resulting from the non-linear time-history analyses when using the natural ground acceleration records.

**Table 9-9 Averaged maximum inter-storey drifts resulting from the non-linear time-history analyses using the four modified ground acceleration records compared to those using the four natural ground acceleration records for structures 4/6/E/T, 4/6/3101/T and 4/6/1170/T**

| *Level* | *4/6/E/T* | *4/6/3101/T* | *4/6/1170/T* |
|---|---|---|---|
| | *Modified / Natural* | *Modified / Natural* | *Modified / Natural* |
| *1* | 1.32 | 1.33 | 1.51 |
| *2* | 1.31 | 1.32 | 1.41 |
| *3* | 1.25 | 1.28 | 1.25 |
| *4* | 1.22 | 1.25 | 1.23 |

*9.3.2.3 Maximum displacements*

Table 9.10 shows the averaged displacements and standard deviations of structures 4/6/E/T, 4/6/3101/T and 4/6/1170/T for the eight ground acceleration records considering P-delta effects. Structure 4/6/E/T has a greater averaged maximum displacement than structures 4/6/3101/T and 4/6/1170/T by 14% and 5% respectively at level 1.

**Table 9-10 Averaged maximum displacements of structures 4/6/E/T, 4/6/3101/T and 4/6/1170/T with P-delta effects**

| Level | *4/6/E/T* | | *4/6/3101/T* | | *4/6/1170/T* | |
|---|---|---|---|---|---|---|
| | Average (m) | Std Deviation (m) | Average (m) | Std Deviation (m) | Average (m) | Std Deviation (m) |
| 1 | 0.068 | 0.022 | 0.060 | 0.025 | 0.065 | 0.041 |
| 2 | 0.132 | 0.042 | 0.122 | 0.047 | 0.145 | 0.075 |
| 3 | 0.179 | 0.056 | 0.180 | 0.065 | 0.207 | 0.086 |
| 4 | 0.209 | 0.067 | 0.224 | 0.077 | 0.236 | 0.090 |

Figure 9.6 shows a comparison between the averaged envelope of displacements found in the non-linear time-history analyses with P-delta effects for structures 4/6/E/T, 4/6/3101/T and 4/6/1170/T with the corresponding values found for the combined modal analysis with P-delta actions. The averaged envelope displacements of structures 4/6/E/T, 4/6/3101/T and 4/6/1170/T show a difference of 24%, 9% and 18% at level 1, when compared to the corresponding value from the modal analysis with P-delta effects.

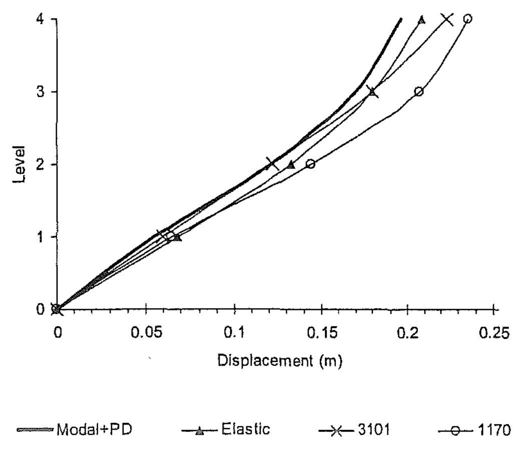



**Figure 9-6 Averaged maximum displacements of structures 4/6/E/T, 4/6/3101/T and 4/6/1170/T for non-linear time-history analyses considering P-delta effects compared to modal displacements plus P-delta effects**

Figure 9.7 illustrates the scatter of maximum displacements for the eight ground acceleration records and the modal plus P-delta actions deformed shape of structures 4/6/3101/T and 4/6/1170/T. The modified ground acceleration record El Centro EW gives the most critical averaged maximum displacements for structures 4/6/3101/T and 4/6/1170/T with 2 and 2.8 times the displacement at level 1 from the modal analysis with P-delta effects. These values have a great inference in the standard deviation.

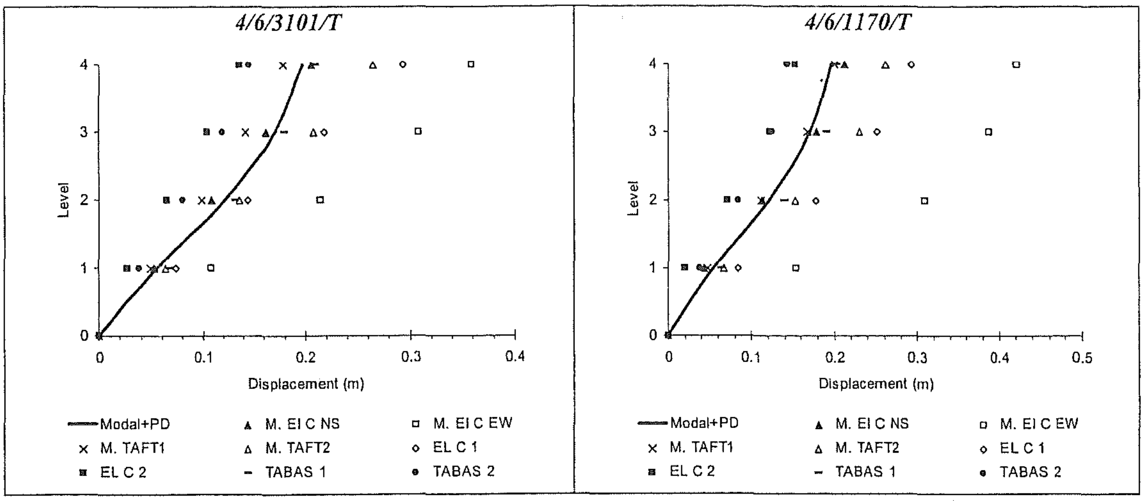



**Figure 9-7 Maximum displacements of structures 4/6/3101/T and 4/6/1170/T for eight ground acceleration records**

### *9.3.2.4 Beam rotations*

Table 9.11 shows the beam rotation demand from the equivalent static and modal analyses and the averaged maximum beam rotations and standard deviations from the non-linear time-history analyses of structures 4/6/E/T, 4/6/3101/T and 4/6/1170/T calculated as described in section 6.3.4.1.

**Table 9-11 Equivalent static and modal analysis beam rotation demand and average of maximum beam rotations and standard deviations of structures 4/6/E/T, 4/6/3101/T and 4/6/1170/T using eight ground acceleration records**

| *Level* | *Eq. static rotations (rad)* | *Modal rotations (rad)* | *4/6/E/T* | | *4/6/3101/T* | | *4/6/1170/T* | |
|---|---|---|---|---|---|---|---|---|
| | | | *Average rot. (rad)* | *Std. Dev. (rad)* | *Average rot. (rad)* | *Std. Dev. (rad)* | *Average rot. (rad)* | *Std. Dev. (rad)* |
| *1* | 0.0189 | 0.0193 | 0.0179 | 0.0067 | 0.0158 | 0.0074 | 0.0151 | 0.0081 |
| *2* | 0.0234 | 0.0230 | 0.0153 | 0.0052 | 0.0167 | 0.0065 | 0.0165 | 0.0037 |
| *3* | 0.0180 | 0.0168 | 0.0118 | 0.0046 | 0.0142 | 0.0048 | 0.0091 | 0.0026 |
| *4* | 0.0105 | 0.0094 | 0.0069 | 0.0045 | 0.0002 | 0.0000 | 0.0002 | 0.0000 |

Figure 9.8 shows a comparison between the averaged envelope of displacements found in the non-linear time-history analyses with P-delta effects for structures 4/6/E/T, 4/6/3101/T and 4/6/1170/T with the corresponding values found for the combined modal and equivalent static analyses with P-delta actions. For the 3 structures, all the averaged maximum beam rotation values from the non-linear time-history analyses are below the beam rotations for the modal and equivalent static analyses with P-

delta effects. It should be noted that the design rotations from the modal and equivalent static analyses were scaled to match the 2.5% inter-storey drift expected in the non-linear analyses. The rotations at the top floor of structures 4/6/3101/T and 4/6/1170/T are reduced because column hinging was allowed in the design at this level.

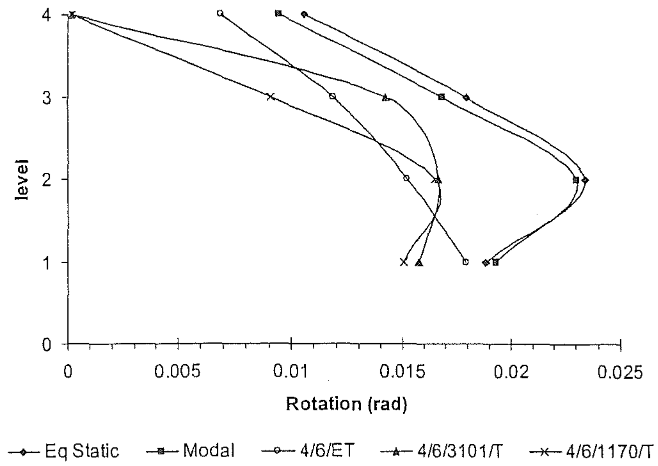



**Figure 9-8 Equivalent static and modal analysis beam rotation demand and average of maximum beam rotations of structures 4/6/E/T, 4/6/3101/T and 4/6/1170/T from eight ground acceleration records**

Figure 9.9 shows the maximum beam rotations of structures 4/6/3101/T and 4/6/1170/T for the non-linear time-history analyses using the eight ground acceleration records. The modified El Centro EW and El Centro 1 records are the only two that show maximum beam rotations that exceed the beam rotations from the equivalent static and modal analyses for both structures. At level one, the equivalent static beam rotation is exceeded by 60% for the non-linear time-history analyses of structures 4/6/3101/T and 4/6/1170/T using the modified El Centro EW ground acceleration record. The scatter of the beam rotations at level 1 of both structures is evident.

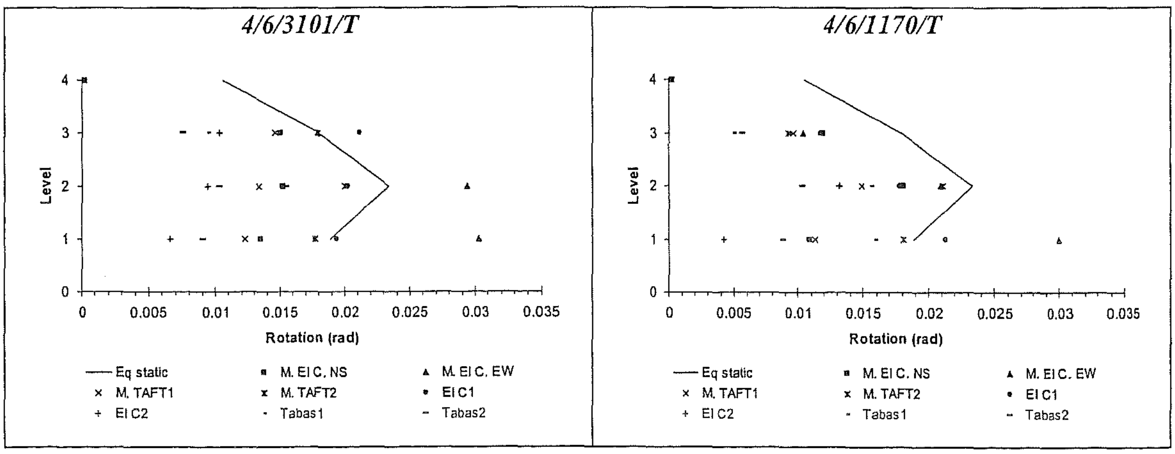



**Figure 9-9 Maximum beam rotations of structures 4/6/3101/T and 4/6/1170/T from non-linear time-history analyses**

Table 9.12 shows the average maximum beam section ductility for the eight earthquake records of structures 4/6/E/T, 4/6/3101/T and 4/6/1170/T. The 3 structures show a maximum averaged beam section ductility of 20 that is well below the corresponding value for the equivalent static and modal analyses.

**Table 9-12 Average of maximum beam section rotation ductilities**

| Level | *Eq static* | Modal | 4/6/E/T | 4/6/3101/T | 4/6/1170/T |
|---|---|---|---|---|---|
| | $S_\mu$ | $S_\mu$ | $S_\mu$ | $S_\mu$ | $S_\mu$ |
| 1 | 22.8 | 23.4 | 21.7 | 19.1 | 18.2 |
| 2 | 32.2 | 31.7 | 21.0 | 22.9 | 22.7 |
| 3 | 34.9 | 32.7 | 23.0 | 27.6 | 17.7 |
| 4 | 20.5 | 18.4 | 13.4 | 0.4 | 0.4 |

### *9.3.2.5 Interior column rotations*

Table 9.13 shows the yield rotation, $\theta_y$, the averaged maximum rotation for the eight non-linear time-history analyses, its standard deviation and the averaged maximum curvature section ductility, $S_\mu$, for an interior column of structures 4/6/3101/T and 4/6/1170/T respectively. Structure 4/6/3101/T shows a column rotation ductility of 10 at the base and top floor where the column was designed for plastic behaviour. For structure 4/6/3101/T, the maximum averaged section ductility is of 3.2, below the beam of the third level. The greatest column rotation ductility for structure 4/6/1170/T is 11.3 at ground level.

**Table 9-13 Interior columns design yield rotation, averaged maximum rotation and standard deviation for eight ground acceleration records for structures 4/6/3101/T and 4/6/1170/T**

| *level* | *4/6/3101/T* | | | | *4/6/1170/T* | | | |
|---|---|---|---|---|---|---|---|---|
| | $\theta_y$ *(rad)* | *Avg. rot (rad)* | *Std. dev. (rad)* | $S_\mu$ | $\theta_y$ *(rad)* | *Avg. rot (rad)* | *Std. dev. (rad)* | $S_\mu$ |
| ***G. L.*** | 0.0013 | 0.0128 | 0.0060 | 10.2 | 0.0013 | 0.0141 | 0.0102 | 11.3 |
| ***1 Below*** | 0.0013 | 0.0012 | 0.0001 | 0.9 | 0.0012 | 0.0030 | 0.0043 | 2.5 |
| ***1 Above*** | 0.0013 | 0.0014 | 0.0001 | 1.1 | 0.0012 | 0.0078 | 0.0041 | 6.4 |
| ***2 Below*** | 0.0014 | 0.0014 | 0.0002 | 1.0 | 0.0012 | 0.0066 | 0.0071 | 5.6 |
| ***2 Above*** | 0.0014 | 0.0012 | 0.0002 | 0.9 | 0.0012 | 0.0023 | 0.0007 | 2.0 |
| ***3 Below*** | 0.0012 | 0.0040 | 0.0037 | 3.2 | 0.0011 | 0.0086 | 0.0029 | 7.7 |
| ***3 Above*** | 0.0012 | 0.0014 | 0.0006 | 1.1 | 0.0011 | 0.0015 | 0.0008 | 1.3 |
| ***4 Below*** | 0.0011 | 0.0126 | 0.0043 | 11.4 | 0.0011 | 0.0079 | 0.0018 | 7.3 |

Figure 9.10 shows the column yield rotation and the maximum rotation from the eight non-linear time-history analyses of structure 4/6/3101/T. For the column sections below the beam, without considering the base of the column and the top floor, the maximum section rotation ductility from the non-linear time-history analyses is 10 at the third level when using the modified El Centro EW record.

The column sections above the beams show a maximum rotation ductility of 1.9 at the third level when using the ground acceleration record El Centro 1.

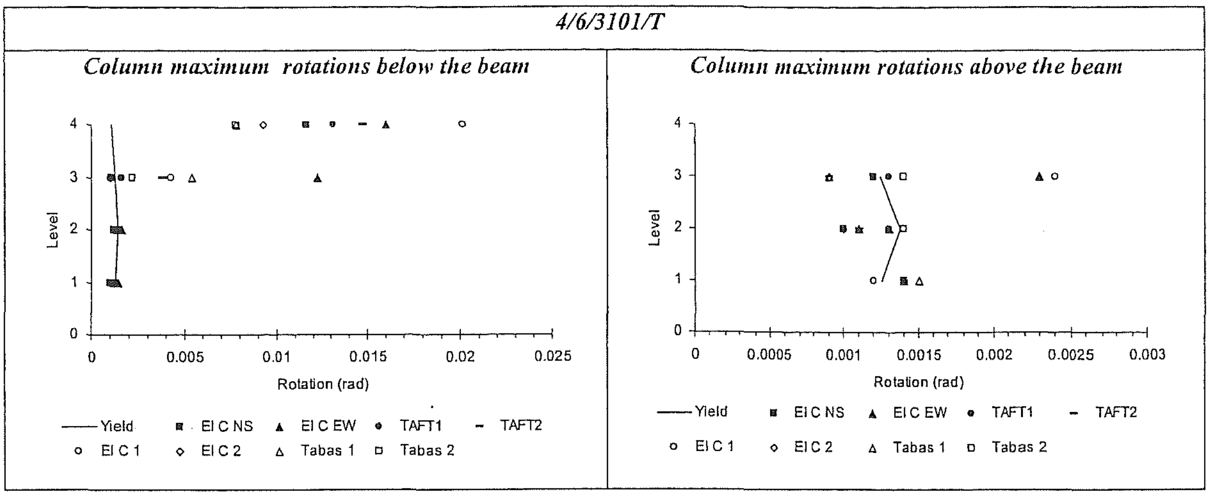



**Figure 9-10 Maximum column rotations for non-linear time-history analyses to structure 4/6/3101/T**

Figure 9.11 shows the column yield rotation and the maximum rotation from the eight non-linear time-history analyses of structure 4/6/1170/T. The maximum section rotation ductility for the column sections below the beam, without considering the top floor and the ground level, is 19 at the second level when using the modified ground acceleration record El Centro EW. For the column sections above the beam, the maximum rotation ductility is 14 at level 1 for the modified ground acceleration record El Centro EW.

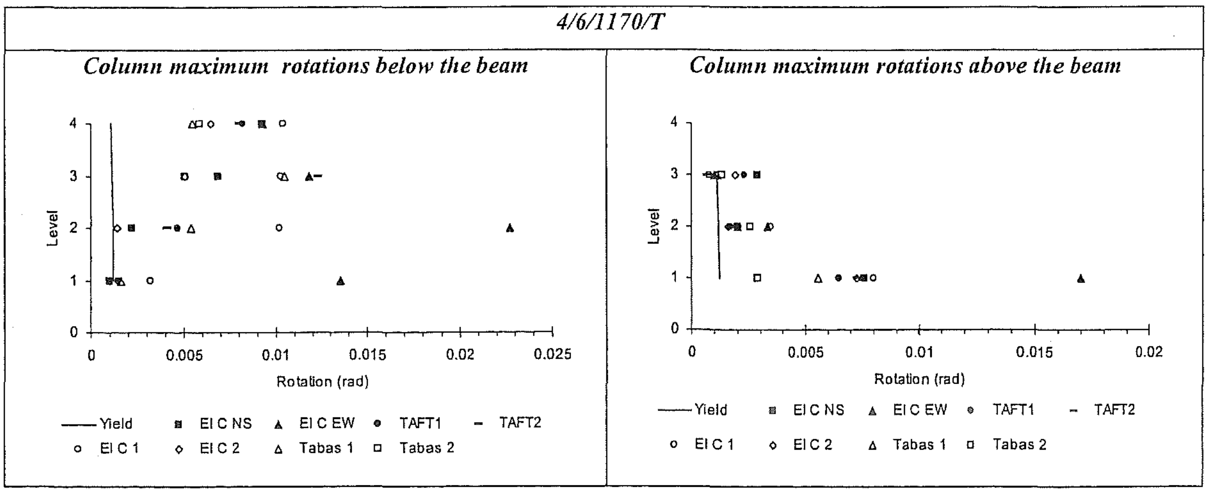



**Figure 9-11 Maximum column rotations for non-linear time-history analyses to structure 4/6/1170/T**

Figure 9.12 compares the average maximum column rotation values for the non-linear time-history analyses of structures 4/6/3101/T and 4/6/1170/T. Structure 4/6/3101/T shows an increase of almost 3 fold in the averaged maximum column rotation at level three below the beam when compared to its respective value above the beam. Structure 4/6/1170/T shows an increase of 2.8 and 5.7 times the averaged maximum column rotation at levels two and three below the beam respectively when compared to its respective value above the beam.

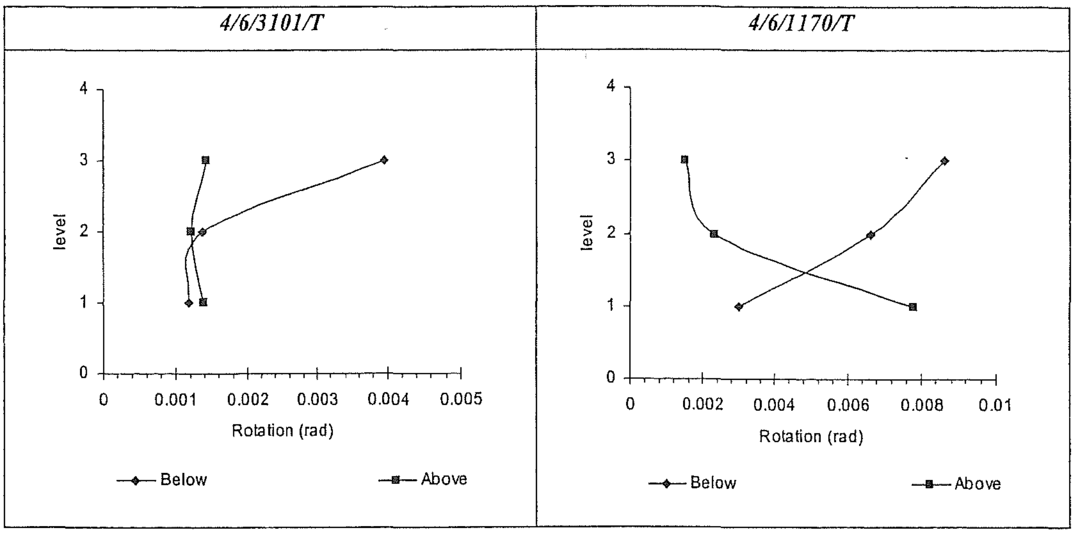



**Figure 9-12 Comparison between average maximum rotations below and above the beam for structures 4/6/3101/T and 4/6/1170/T**

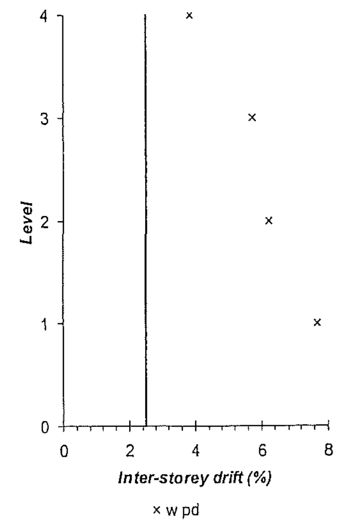
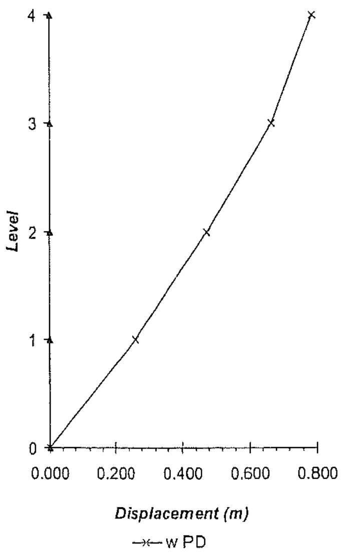
### *9.3.3 Earthquake return period of 2,500 years*

The response of structures 4/6/3101/T and 4/6/1170/T for non-linear time-history analyses using ground acceleration records El Centro 1 and Tabas 1, scaled to simulate a 2,500 years return period earthquakes as described in section 5.13.3, was obtained. Table 9.14 summarizes the response of structure 4/6/3101/T and figure 9.13 shows the response of structure 4/6/1170/T when using ground acceleration record El Centro 1. Table 9.22 shows the results for non-linear time-history analyses of structures 4/6/3101/T and 4/6/1170/T using the ground acceleration record Tabas 1. All analyses were run considering P-delta actions.

With the El Centro 1 record the analysis indicated that collapse occurred for structure 4/6/1170/T. With structure 4/6/3101/T no collapse was indicated. However, as the inter-storey drifts reached 8%and the section curvature in the beams reached 84 times the yield curvature, it is unlikely that collapse would have been avoided. Deformations of this order would have caused strength degradation, and this was not modelled in the analyses.

The maximum inter-storey drifts 3.66% and 4.66 % for structures 4/6/3101/T and 4/6/1170/T respectively when using the ground acceleration record Tabas 1 scaled to a 2,500 years return period earthquake indicate that the collapse of the structure could be avoided if its elements are well detailed. Beam rotations are critical at level two of both models reaching values of 0.0357 and 0.0316 radians which represent a section rotation ductility of 49 and 43 respectively.

**Table 9-14 Results for non-linear time-history analysis of structure 4/6/3101/T using El Centro 1 ground acceleration record scaled to match a 2,500 years return period earthquake**

| *Record "El Centro 1"* | | | |
|---|---|---|---|
| *4/6/3101/T* | | | |
| *Inter-storey drifts* |  | *Maximum displacements* |  |
| | *Interior column rotation (rad)* | *Level* | *Centre span beam rotations (rad)* |
| *G.L.* | 0.0634 | *1* | 0.0588 |
| *1 Below* | 0.0174 | *2* | 0.0612 |
| *1 Above* | 0.0069 | *3* | 0.0430 |
| *2 Below* | 0.0020 | *4* | 0.0003 |
| *2 Above* | 0.0017 | | |
| *3 Below* | 0.0153 | | |
| *3 Above* | 0.0017 | | |
| *4 Below* | 0.0348 | | |

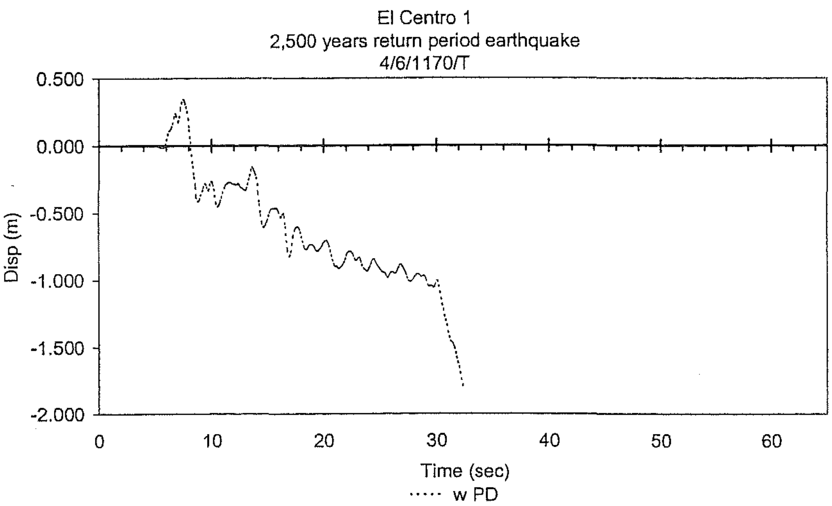
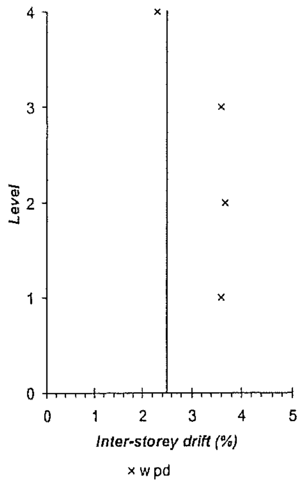
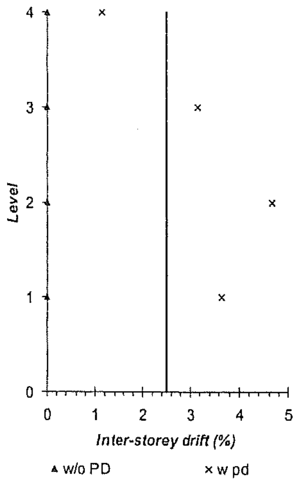
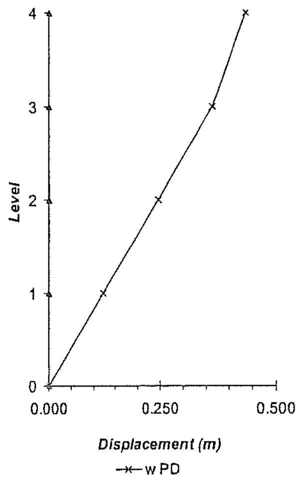
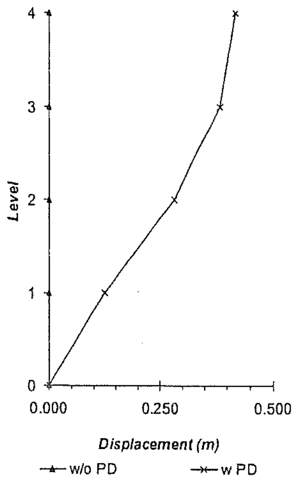



**Figure 9-13 Top floor displacement versus time for non-linear time-history analysis using ground acceleration record El Centro 1 scaled to a 2,500 years return period earthquake of structure 4/6/1170/T.**

**Table 9-15 Results for non-linear time-history analysis of structure 4/6/3101/T and 4/6/1170/T using Tabas 1 ground acceleration record scaled to match a 2,500 years return period earthquake**

| *Record "Tabas 1"* | | |
|---|---|---|
| | *4/6/3101/T* | *4/6/1170/T* |
| *Inter-storey drifts* |  |  |
| *Maximum displacements* |  |  |
| ***Level*** | ***Centre span beam rotations (rad)*** | ***Centre span beam rotations (rad)*** |
| *1* | 0.0352 | 0.0290 |
| *2* | 0.0357 | 0.0316 |
| *3* | 0.0234 | 0.0130 |
| *4* | 0.0003 | 0.0002 |
| | ***Interior column rotation (rad)*** | ***Interior column rotation (rad)*** |
| ***G.L.*** | 0.0281 | 0.0291 |
| ***1 Below*** | 0.0014 | 0.0078 |
| ***1 Above*** | 0.0015 | 0.0177 |
| ***2 Below*** | 0.0016 | 0.0161 |
| ***2 Above*** | 0.0016 | 0.0052 |
| ***3 Below*** | 0.0121 | 0.0168 |
| ***3 Above*** | 0.0046 | 0.0032 |
| ***4 Below*** | 0.0205 | 0.0103 |

# 10. CONCLUSIONS

The following conclusions were obtained from the analyses performed in this work:

1. Structures designed to the minimum strength or designed with the additional strength required to comply with minimum steel requirements:

As seen when comparing the response of structures 6/6/E/T and 6/6/E/T* (see section 6.3), the additional strength given to beams in the upper levels to comply with the minimum steel requirements makes the inter-storey drifts in the lower half of the structure increase by up to 25% when P-delta effects are considered. Because of the additional strength, the upper levels contribute less to the energy dissipation and show little maximum inter-storey drifts. The structure that complies with minimum reinforcement steel requirements shows a greater difference when its averaged envelope of displacements from the non-linear time-history analyses is compared to the shape of its first vibration mode than the structure designed to the minimum strength requirements. The averaged shape of the envelope of displacements for the structure designed to the minimum strength requirements is almost linear. This is illustrated in figure 10.1 where the displacement envelope from the non-linear time-history analyses was averaged and then normalized to the top floor displacement. This figure also shows the first mode shape of the 6 storey building normalized to the displacement of the top floor. The shape of the displacement envelope should be carefully considered when designing a structure using an equivalent static analysis.

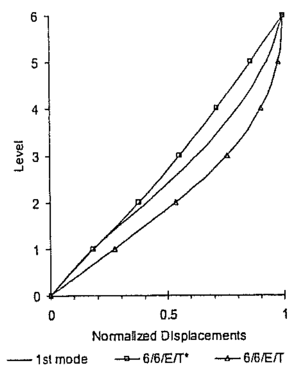



**Figure 10-1 First mode of vibration shape and normalized averaged displacement envelope from non-linear time-history analyses to structures 6/6/E/T and 6/6/E/T***

2. Structures designed to different ductility levels:

From the 6 storey buildings designed with different ductilities, a tendency is observed in the envelope displacement shapes when comparing the structures. As the ductility increases, the envelope displacement shape tends to be less linear (see figure 6.7). This difference is more marked when P-delta effects are considered. This tendency is shown in part because as the design ductility is reduced, the number of beams to which the strength has to be increased to comply with minimum steel requirements decreases. For the structure 6/1/E/T all the beams are designed to the minimum required strength. This is also reflected in the averaged maximum inter-storey drifts. At storey one, the structure designed to a ductility of 6 has the greatest averaged maximum inter-storey drift while at the $6^{th}$ storey the most critical value comes from the structure designed to a ductility of 1.

By comparing the inter-storey drifts coming from the displacement envelope of the non-linear time-history analyses of the structures designed to different ductility levels with the real inter-storey drifts, it could be seen that the change in assessing the forces to account for P-delta effects in a modal or an equivalent static analyses using the maximum displacements or the maximum inter-storey drifts would be negligible for the lower levels which are the most critical.

3. Effect of the choice of hysteretic model:

The maximum inter-storey drifts and maximum displacements of a six and a twelve storey structures were compared when each of these structures were modelled with a Takeda and a bilinear hysteretic model. For the six and the twelve storey structures when P-delta effects are not considered, the difference of the maximum inter-storey drifts from the structures modelled with a Takeda hysteretic model show a negligible difference when compared to the corresponding values of the structures modelled using a bilinear hysteretic model. If P-delta effects are considered, a maximum inter-storey drift greater by up to 9% is shown for the 6 storey structure using the bilinear hysteretic model over the corresponding values when it is modelled using a Takeda hysteretic model. This value goes up to 40% for the 12 storey structure.

4. Base shear:

The averaged maximum base shear from the non-linear time-history analyses of all structures considering P-delta effects was around twice the base shear from the modal and equivalent static analyses with the addition of P-delta forces. This increment comes from the difference in shear distribution in the elastic and non-linear analyses, the strength increase in the beams due to the strength reduction factor, the minimum longitudinal reinforcement requirements and the strain hardening of the reinforcement. As shown in appendix C, the damping forces are having a great influence in the base shear and the different damping models and its effects in the analyses should be further studied. This difference in the base shear from the elastic analysis should be accounted for and the beams and columns designed accordingly.

5. Maximum inter-storey drifts:

The averaged maximum inter-storey drifts of all structures, with the three different types of columns are always less than 2.5% when P-delta effects are included. The averaged maximum inter-storey drifts of the structures modelled with columns designed for a high protection against plastic hinges are closer to the inter-storey drift limit set for the elastic analyses of each structure than to the 2.5% limit set for the inelastic analyses. The structures modelled with columns with a low protection against plastic hinges show an averaged maximum inter-storey drift closer to 2.5%. If structures would have used a bilinear hysteretic model instead of a more realistic Takeda model, the maximum inter-storey drifts would be expected to be greater than the 2.5%.

The maximum inter-storey drifts from the non-linear time-history analyses showed great scatter and for every structure, at least one record exceeded the 2.5% limit. The 12 storey A structure and the 4 storey structure showed for one record a maximum inter-storey drifts of 5.4% and 4.4% respectively for the models with columns designed to a low protection against the formation of plastic hinges. A structure that shows inter-storey drifts of this magnitude would only avoid collapse with proper detailing and construction.

6. Beam rotations:

The averaged maximum beam rotation ductilities for all structures were not greater than 27 and in every structure a beam rotation ductility of at least 20 was found. The difference in the averaged maximum beam rotations between the structures with different column characteristics did not vary significantly. In any case the averaged maximum beam rotation exceeded the design beam rotation from the modal or the equivalent static analyses.

7. Interior column rotations

For the structures with columns designed with a high protection against the formation of plastic hinges the averaged maximum rotation ductility was in all cases close to 1. The most critical averaged maximum column rotation ductility for the 6 storey structure was 1.95 and for the two 12 storey structures and the 4 storey structure, the averaged maximum column rotation ductility never exceeded 1.1.

For the structures designed with columns with a low protection against the formation of plastic hinges, the maximum column rotation ductility was always less than 11.

8. Structures with elastic columns above the base and structures with columns designed for a high protection against plastic hinge formation:

At all cases, the results from the non-linear time-history analyses of structures modelled with elastic columns above the base compared to those for the structures modelled with columns with a high protection against plastic hinges were found to have small variation. This would verify that enough strength has been given to the columns to avoid plastic hinging.

9. Structures with columns designed for a high protection against plastic hinge formation (3101) and structures with columns designed for a low protection against plastic hinge formation (1170):

Even though inelastic behaviour was observed in the columns of structures designed with a low protection against plastic hinge formation, the beam rotation did not decrease significantly when compared to the structures designed with a high protection against plastic hinge formation.

Structures 6/6/1170/T, 12A/6/1170/T and 4/6/1170/T showed a decrease in the averaged maximum base shear of 9%, 4% and 2% respectively when compared to the corresponding values for structures 6/6/3101/T, 12A/6/3101/T and 4/6/3101/T.

For the six storey structure, the different column designs did not show a significant variation on the averaged maximum inter-storey drifts. Structures 12A/6/1170/T and 4/6/1171/T showed an increase in the averaged maximum inter-storey drift that went from 1.70% and 1.85% respectively to 2.23% and 2.38% for the structures 12A/6/3101/T and 4/6/3101/T.

10. P-delta effects

For the structures modelled with the columns with a high protection against plastic hinges, the inclusion of P-delta effects in the analyses indicated an increase in the averaged maximum inter-storey drifts of 12% for the 4 and 6 storey structures and a 25% for the 12 storey A and B structures from the non-linear time-history analyses. For the structures modelled with the columns with a low protection against plastic hinges, the inclusion of P-delta effects increased the averaged maximum inter-storey drifts by 15%, 51% and 23% for the 6, 12A and 4 storey structures.

The base shear showed an increase between 9% and 14% for the non-linear time-history analyses that included P-delta effects when compared to those that did not. This increase had little variation in all the structures, including the models with different column designs.

It is clear that P-delta effects have significant effects even for the four storey structures. Since P-delta effects are always present in real structures, it is suggested that they should be considered in the design and analysis of all structures.

11. Ground acceleration records modified to match the design spectra across the whole range of periods compared to natural ground acceleration records:

The use of modified ground acceleration records did not have a great influence in the averaged maximum base shear of the structures. The most critical cases were the 6 and 4 storey structures that showed an increase in its averaged maximum base shear of 11% and 9% for the non-linear time-history analyses using the modified ground acceleration records when compared to the corresponding value when using the natural ground acceleration records. The structures 12A and 12B showed an increase of 2% and 6% respectively when using modified ground acceleration records compared to the corresponding results using the natural ground acceleration records.

The averaged maximum inter-storey drift showed a great increase when using the modified ground acceleration records compared to the results using the natural ground acceleration records. This increase was particularly marked for the structures designed with columns with a low protection against the formation of plastic hinges showing an increase of 62%, 280% and 50% for the 6, 12A and 4 storey structures. The structures designed with a high protection against plastic hinges showed an increase of 42%, 84%, 77% and 33% for structures 6/6/3101/T, 12A/6/3101/T, 12B/6/3101/T and 4/6/3101/T respectively when the averaged maximum inter-storey drift using the modified ground acceleration records are compared to the corresponding values using the natural ground acceleration records.

The modified ground acceleration records showed a large scatter in the maximum inter-storey drifts, maximum displacements and maximum beam and column rotations.

12. Earthquake with a return period of 2,500 years return period

The six storey structures 6/6/3101/T and 6/6/1170/T performed fine for the 2,500 years return period earthquakes showing a maximum inter-storey drift of 4.2% and 4.8% respectively for the ground acceleration record El Centro 1. Structures 12A/6/3101/T and 12A/6/1170/T collapsed showing a soft first storey when subject to the El Centro 1 ground acceleration record. Structure 4/6/1170/T also collapsed under the El Centro 1 ground acceleration record and structure 4/6/3101/T showed a maximum inter-storey drift of 8% which would have lead a real structure to collapse. It should be noted that strength degradation was not modelled but it would be expected in real structures subjected to ground accelerations of this magnitude.

P-delta effects were shown to have a significant influence even in the response of the four and six storey structures. In general, the structures studied performed in a satisfactory manner due to the use of a more realistic hysteretic model. This good performance was apparent for the global behaviour of the structure as well as for its individual elements but further studies should be made to assess its performance to the maximum credible event described by the Standard.

# 11. SUGGESTIONS FOR FURTHER RESEARCH

From this work it was found that further research should be undertaken in the following areas:

- Investigate the effects of different damping models on the response of ductile frames. The damping models used in these studies indicate the presence of significant damping forces and their influence to the response of the structures is not yet fully understood.

- A limited number of non-linear time-history analyses were run using a 2,500 year return period earthquake in which the structures developed a soft storey. The response of structures to the maximum credible event considering the effects of strength degradation should be further studied.

- The family scaling factor, $k_2$, for the ground acceleration records described in the NZS draft 1170.5 was not included in the scaling of the ground acceleration records in this work. It should be investigated how the inclusion of this scaling factor would have affected the results and assessed how conservative would the results obtained using ground acceleration records scaled in this manner would turn out to be.

- The scatter of the results from the non-linear time-history analyses should be investigated and its influence when designing structures by non-linear time-history analyses using a given number of ground acceleration records.

- Soil-structure interaction was not included in this work. Research considering P-delta effects, a realistic hysteretic model and soil structure interaction should be undertaken for the different zones in New Zealand, particularly those with softer soils.

- The high shears resulting from the non-linear time-history analyses should be studied in more detail and the shear design process for beams and columns revised. It would also be interesting to include the effects of shear deformation and the effects of member elongation to the analyses.

- In this project, the minimum longitudinal steel reinforcement ratio was not considered in the design of columns. Studies should be made to check the effect it would have in the rotation of the columns, especially those designed with a low protection against the formation of plastic hinges.

## 12. REFERENCES


Bernal, D. (1987). "Amplification Factors for Inelastic Dynamic P-delta Effects in Earthquake Analysis." *Earthquake Engineering and Structural Dynamics*, 15, 631-651.

Bull, D. K., Brunsdon, D. R., Cement & Concrete Association of New Zealand., and New Zealand Concrete Society. (1998). *Examples of concrete structural design to New Zealand Standard 3101*, Cement & Concrete Association of New Zealand ; NZCS, Wellington, N.Z.

Carr, A. J. (2003). *A program for inelastic dynamic analysis*, Christchurch.

Chung, B. T. (1993). *The influence of P-Delta effects on the seismic design of structures*, Dept. of Civil Engineering University of Auckland, Auckland, N.Z.

Chung, B. T., Fenwick, R. C., and Davidson, B. J. (1991). *Dynamic behaviour of multi-storey buildings*, Department of Civil Engineering University of Auckland, Auckland, N.Z.

Fenwick, R. C., and Davidson, B. J. (1991). *The seismic response of multi-storey buildings*, Dept. of Civil Engineering University of Auckland, Auckland, N.Z.

Fenwick, R. C., and Davidson, B. J. (1994). "A critical assessment of the seismic design forces and stiffness requirements in the Loadings Standard, NZS4203-1992." *Proceedings NZSEE Technical Conference*, 102-110.

Fenwick, R. C., Davidson, B. J., and Chung, B. T. (1992). "P-delta Actions in Seismic Resistant Structures." *Bulletin of the New Zealand National Society for Earthquake Engineering*, 25(1), 56-69.

Fenwick, R. C., Lau, D., and Davidson, B. J. (2002). "A comparison of seismic design requirements in the New Zealand Loadings Standard with other major design codes." *NZSEE Bulletin*, 35(4), 190-203.

Fenwick, R. C., Tankut, A. T., and Thom, C. W. (1981). *The deformation of reinforced concrete beams subjected to inelastic cyclic loading. Experimental results*, Department of Civil Engineering University of Auckland, Auckland, N. Z.

Gill, W. D. (1979). *Ductility of rectangular reinforced concrete columns with axial load*, Dept. of Civil Engineering University of Canterbury, Christchurch, N.Z.

Hollings, J. P. (1969). "Reinforced Concrete Seismic Design." *Bulletin of the New Zealand National Society for Earthquake Engineering*, 2(3), 217-250.

Jury, R. D. (1978). *Seismic load demands on columns of reinforced concrete multistorey frames*, Christchurch,.

Kelly, T. E. (1974). *Some seismic design aspects of multistorey concrete frames : a report submitted in partial fulfilment of the requirements for the degree of Master of Engineering [Civil Engineering] at the University of Canterbury, Christchurch, New Zealand.*

Kelly, T. E. (1977). "Uniaxial Dynamic Analysis of a Six Storey Reinforced Concrete Framed Structure." *Bulletin of the New Zealand National Society for Earthquake Engineering*, 10(1), 37-51.

Lindup, G. H. (1975). *Seismic demands on columns of reinforced concrete multistorey frames : a report submitted in partial fulfilment of the requirements for the degree of Master of Engineering at the University of Canterbury, Christchurch, New Zealand.*

Moss, P. J., and Carr, A. J. (1980). "The Effects of Large Displacements on the Earthquake Response of Tall Concrete Frame Structures." *Bulletin of the New Zealand National Society for Earthquake Engineering*, 13(4), 317-328.

Munro, I. R. M. (1976). *Seismic behaviour of reinforced concrete bridge piers : a report submitted in partial fulfilment of the requirements for the degree of Master of Engineering at the University of Canterbury, Christchurch, New Zealand*, University of Canterbury, Christchurch.

Pampanin, S., Christopoulos, C., and Priestley, M. J. N. (2002). *Residual deformations in the performance-based seismic assessment of frame structures*, IUSS Press, Pavia, Italy.

Park, R. (1995). "The Paulay years. Recent developments on the earthquake response of tall concrete frame structures." *Bulletin of the New Zealand National Society of Earthquake Engineering*.

Park, R. (2001). "Improving the resistance of structures to earthquakes." *Bulletin of the New Zealand National Society for Earthquake Engineering*, 34(1), 1-40.

Park, R. (2003). "Some controversial aspects of the seismic design of reinforced concrete building structures." *Bulletin of the New Zealand National Society for Earthquake Engineering*, 36(3), 165-188.

Park, R., and Paulay, T. (1975). *Reinforced concrete structures : R. Park and T. Paulay*, Wiley, New York.

Paulay, T. (1977). "Columns - Evaluation of Actions." *Bulletin of the New Zealand National Society for Earthquake Engineering*, 10(2), 85-93.

Paulay, T. (1979). "Developments in the design of ductile reinforced concrete frames." *Bulletin of the New Zealand National Society for Earthquake Engineering*, 12, 33-48.

Paulay, T. (1980). "Deterministic Design Procedure for Ductile Frames in Seismic Areas in: "Reinforced Concrete Structures Subjected to Wind and Earthquake Forces"." *ACI publication*, SP-63, 357-381.

Paulay, T. (1988). "Seismic Design in Reinforced Concrete: The State of the Art in New Zealand." *Bulletin of the New Zealand National Society for Earthquake Engineering*, 21(3), 208-232.

Paulay, T., and Priestley, M. J. N. (1992). *Seismic design of reinforced concrete and masonry buildings*, Wiley, New York, N.Y.

Priestley, M. J. N. (2003). *Myths and fallacies in earthquake engineering, revisited*, IUSS Press, Pavia, Italy.

Priestley, M. J. N., and Amaris, A. D. (2002). *Dynamic amplification of seismic moments and shear forces in cantilever walls*, IUSS Press, Pavia, Italy.

Row, D. G. (1973). *The effects of skew seismic response on reinforced concrete frames : a report submitted in partial fulfilment of the requirements for the degree of Master of Engineering at the University of Canterbury, Christchurch, New Zealand.*

Standards New Zealand. (1992). *Code of practice for general structural design and design loadings for buildings, known as the loadings standard*, Standards New Zealand, Wellington [N.Z.].

Standards New Zealand (1995). *Concrete structures standard*, Standards New Zealand, Wellington [N.Z.].

Tompkins, D. N. (1980). *The seismic response of reinforced concrete multistorey frames*, Dept. of Civil Engineering University of Canterbury, Christchurch.

Zhao, W., Davidson, B. J., and Fenwick, R. C. (2001). *The influence of P-delta effects on seismic response of structures with various hysteretic models*, Dept. of Civil and Environmental Engineering University of Auckland, Auckland, N.Z.

## APPENDIX A. CHARACTERISTICS OF STRUCTURES

For a general description of each model for all structures see section 5.1. For the beam and columns cross sections see section 5.12.

### *A.1 Six Storey Building*

The post-yield stiffness *(r)* used for all the members in the six storey structures is ***0.007***.

**Table A-1 Six storey structure beams effective moment of inertia**

| | *Beams effective moment of inertia ($I_{eff}$) ($m^4$)* | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | *Model* | | | | | | | |
| *Level* | *6/6/E/T** | *6/6/E/T* | *6/6/3101/T* | *6/6/1170/T* | *6/6/E/B* | *6/4/E/T* | *6/2/E/T* | *6/1/E/T* |
| *1* | 0.00866 | 0.00866 | 0.00866 | 0.00866 | 0.00866 | 0.00866 | 0.00866 | 0.00866 |
| *2* | 0.00866 | 0.00866 | 0.00866 | 0.00866 | 0.00866 | 0.00866 | 0.00866 | 0.00866 |
| *3* | 0.00866 | 0.00866 | 0.00866 | 0.00866 | 0.00866 | 0.00866 | 0.00866 | 0.00866 |
| *4* | 0.00866 | 0.00866 | 0.00866 | 0.00866 | 0.00866 | 0.00866 | 0.00866 | 0.00866 |
| *5* | 0.00866 | 0.00866 | 0.00866 | 0.00866 | 0.00866 | 0.00866 | 0.00866 | 0.00866 |
| *6* | 0.00866 | 0.00866 | 0.00866 | 0.00866 | 0.00866 | 0.00866 | 0.00866 | 0.00866 |

*Notes:*

*The gross moment of inertia was calculated using half the width of the flange.*

*For details on the section dimensions, see section 5.12*

$I_{gross} = 0.4\ I_{eff}$

**Table A-2 Six storey structure beam nominal moments at column face after redistribution**

| | *Beam nominal moments at column face after redistribution (kN m)* | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | *Model* | | | | | | | |
| *Level* | *6/6/E/T** | *6/6/E/T* | *6/6/3101/T* | *6/6/1170/T* | *6/6/E/B* | *6/4/E/T* | *6/2/E/T* | *6/1/E/T* |
| *1* | 464.3 | 464.3 | 464.3 | 464.3 | 464.3 | 625.0 | 1038.8 | 1917.7 |
| *2* | 452.3 | 452.3 | 452.3 | 452.3 | 452.3 | 610.9 | 1024.2 | 1902.3 |
| *3* | 372.4 | 372.4 | 372.4 | 372.4 | 372.4 | 509.6 | 878.7 | 1645.0 |
| *4* | 286.2 | 372.4 | 372.4 | 372.4 | 372.4 | 396.1 | 701.6 | 1308.1 |
| *5* | 192.8 | 372.4 | 372.4 | 372.4 | 372.4 | 396.1 | 484.2 | 890.4 |
| *6* | 89.0 | 372.4 | 372.4 | 372.4 | 372.4 | 396.1 | 484.2 | 407.1 |

*Notes:*

*Since these are moments after redistribution, they are along the three spans at each level.*

$Mn = M^*/\phi$ ; *where* $\phi = 0.85$

*The over-strength factor used for column design is 1.1.*

**Table A-3 Six storey interior and exterior columns effective moment of inertia**

| Level | *Interior and exterior columns effective moment of inertia ($m^4$)* — *Model*: 6/6/E/T* | 6/6/E/T | 6/6/3101/T | 6/6/1170/T | 6/6/E/B | 6/4/E/T | 6/2/E/T | 6/1/E/T |
|---|---|---|---|---|---|---|---|---|
| *1* | 0.00615 | 0.00615 | 0.00615 | 0.00615 | 0.00615 | 0.00615 | 0.00615 | 0.00615 |
| *2* | 0.00615 | 0.00615 | 0.00615 | 0.00615 | 0.00615 | 0.00615 | 0.00615 | 0.00615 |
| *3* | 0.00615 | 0.00615 | 0.00615 | 0.00615 | 0.00615 | 0.00615 | 0.00615 | 0.00615 |
| *4* | 0.00615 | 0.00615 | 0.00615 | 0.00615 | 0.00615 | 0.00615 | 0.00615 | 0.00615 |
| *5* | 0.00615 | 0.00615 | 0.00615 | 0.00615 | 0.00615 | 0.00615 | 0.00615 | 0.00615 |
| *6* | 0.00615 | 0.00615 | 0.00615 | 0.00615 | 0.00615 | 0.00615 | 0.00615 | 0.00615 |

*Notes:*

*For details on the section dimensions, see section 5.12*

$I_{gross} = 0.4\ I_{eff}$

**Table A-4 Six storey interior column nominal moments at beam face**

| Level | *Interior Column nominal moments at beam face (kN m)* — *Model*: 6/6/E/T* | 6/6/E/T | 6/6/3101/T | 6/6/1170/T | 6/6/E/B | 6/4/E/T | 6/2/E/T | 6/1/E/T |
|---|---|---|---|---|---|---|---|---|
| *G.L.* | 568.8 | 568.8 | 568.8 | 568.8 | 568.8 | 768.9 | 1283.9 | 2360.8 |
| *1* | *Elastic columns* | *Elastic columns* | 737.7 | 450.5 | *Elastic columns* | *Elastic columns* | *Elastic columns* | *Elastic columns* |
| *2* | | | 861.0 | 452.1 | | | | |
| *3* | | | 742.7 | 387.7 | | | | |
| *4* | | | 810.6 | 316.0 | | | | |
| *5* | | | 703.4 | 238.0 | | | | |
| *6* | | | 152.0 | 92.0 | | | | |

Notes:

At ground level, design moments come from Modal analysis plus P-delta induced forces factorised by a factor of 1 / 0.85.

**Table A-5 Six storey exterior column nominal moments at beam face**

| Level | *Exterior Column nominal moments at beam face (kN m)* — *Model*: 6/6/E/T* | 6/6/E/T | 6/6/3101/T | 6/6/1170/T | 6/6/E/B | 6/4/E/T | 6/2/E/T | 6/1/E/T |
|---|---|---|---|---|---|---|---|---|
| *G.L.* | 490.9 | 490.9 | 490.9 | 490.9 | 490.9 | 663.0 | 1108.2 | 2039.0 |
| *1* | *Elastic columns* | *Elastic columns* | 344.8 | 225.3 | *Elastic columns* | *Elastic columns* | *Elastic columns* | *Elastic columns* |
| *2* | | | 439.0 | 226.1 | | | | |
| *3* | | | 398.0 | 193.8 | | | | |
| *4* | | | 442.0 | 206.1 | | | | |
| *5* | | | 415.0 | 230.8 | | | | |
| *6* | | | 135.0 | 192.5 | | | | |

**Table A-6 Six storey interior column design axial forces**

| | *Interior Column design axial forces. (kN)** | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | *Model* | | | | | | | |
| *Level* | *6/6/E/T** | *6/6/E/T* | *6/6/3101/T* | *6/6/1170/T* | *6/6/E/B* | *6/4/E/T* | *6/2/E/T* | *6/1/E/T* |
| *G.L.* | -512.5 | -512.5 | -512.5 | -512.5 | -512.5 | -512.5 | -512.5 | -512.5 |
| *1* | *Elastic columns* | *Elastic columns* | -425.6 | -425.6 | *Elastic columns* | *Elastic columns* | *Elastic columns* | *Elastic columns* |
| *2* | | | -344.1 | -344.1 | | | | |
| *3* | | | -259.8 | -259.8 | | | | |
| *4* | | | -177.1 | -177.1 | | | | |
| *5* | | | -83.6 | -83.6 | | | | |
| *6* | | | -83.6 | -83.6 | | | | |
| ** Positive forces mean tension.* | | | | | | | | |

**Table A-7 Six storey exterior column design axial forces**

| | *Exterior Column design axial forces. (kN)* | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | *Model* | | | | | | | |
| *Level* | *6/6/E/T** | *6/6/E/T* | *6/6/3101/T* | *6/6/1170/T* | *6/6/E/B* | *6/4/E/T* | *6/2/E/T* | *6/1/E/T* |
| *G.L.* | 471.3 | 471.3 | 471.3 | 471.3 | 471.3 | 641.9 | 1184.7 | 2065.4 |
| *1* | *Elastic columns* | *Elastic columns* | 373.9 | 373.9 | *Elastic columns* | *Elastic columns* | *Elastic columns* | *Elastic columns* |
| *2* | | | 284.8 | 284.8 | | | | |
| *3* | | | 219.4 | 219.4 | | | | |
| *4* | | | 155.7 | 155.7 | | | | |
| *5* | | | 82.8 | 82.8 | | | | |
| *6* | | | 82.8 | 82.8 | | | | |
| ** Positive forces mean tension.* | | | | | | | | |

**Table A-8 Six storey interior column reinforcement ratios**

| | *Interior Column reinforcement ratios.* | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | *Model* | | | | | | | |
| *Level* | *6/6/E/T** | *6/6/E/T* | *6/6/3101/T* | *6/6/1170/T* | *6/6/E/B* | *6/4/E/T* | *6/2/E/T* | *6/1/E/T* |
| *G.L.* | 0.0116 | 0.0116 | 0.0116 | 0.0116 | 0.0116 | 0.0173 | 0.0321 | 0.0748 |
| *1* | *Elastic columns* | *Elastic columns* | 0.0170 | 0.0089 | *Elastic columns* | *Elastic columns* | *Elastic columns* | *Elastic columns* |
| *2* | | | 0.0207 | 0.0096 | | | | |
| *3* | | | 0.0180 | 0.0084 | | | | |
| *4* | | | 0.0205 | 0.0072 | | | | |
| *5* | | | 0.0183 | 0.0057 | | | | |
| *6* | | | 0.0080 | 0.0017 | | | | |

**Table A-9 Six storey exterior column reinforcement ratios**

<table>
<tr><td></td><td colspan="8">Exterior Column reinforcement ratios.</td></tr>
<tr><td></td><td colspan="8">Model</td></tr>
<tr><td>Level</td><td>6/6/E/T*</td><td>6/6/E/T</td><td>6/6/3101/T</td><td>6/6/1170/T</td><td>6/6/E/B</td><td>6/4/E/T</td><td>6/2/E/T</td><td>6/1/E/T</td></tr>
<tr><td>G.L.</td><td>0.0175</td><td>0.0175</td><td>0.0175</td><td>0.0175</td><td>0.0175</td><td>0.0237</td><td>0.0410</td><td>0.0632</td></tr>
<tr><td>1</td><td rowspan="6">Elastic columns</td><td rowspan="6">Elastic columns</td><td>0.0126</td><td>0.0091</td><td rowspan="6">Elastic columns</td><td rowspan="6">Elastic columns</td><td rowspan="6">Elastic columns</td><td rowspan="6">Elastic columns</td></tr>
<tr><td>2</td><td>0.0151</td><td>0.0084</td></tr>
<tr><td>3</td><td>0.0133</td><td>0.0069</td></tr>
<tr><td>4</td><td>0.0141</td><td>0.0054</td></tr>
<tr><td>5</td><td>0.0126</td><td>0.0037</td></tr>
<tr><td>6</td><td>0.0080</td><td>0.0020</td></tr>
</table>

## *A.2 Twelve Storey Building A*

The post-yield stiffness *(r)* used for all the members in the twelve storey *A* structures is ***0.004***.

**Table A-10 Twelve storey building A, beams effective moment of inertia**

| | ***Beams effective moment of inertia ($I_{eff}$) ($m^4$)*** | | | |
|---|---|---|---|---|
| | ***Model*** | | | |
| *Level* | *12A/6/E/T* | *12A/6/3101/T* | *12A/6/1170/T* | *12A/6/E/B* |
| *1* | 0.0158 | 0.0158 | 0.0158 | 0.0158 |
| *2* | 0.0158 | 0.0158 | 0.0158 | 0.0158 |
| *3* | 0.0158 | 0.0158 | 0.0158 | 0.0158 |
| *4* | 0.0158 | 0.0158 | 0.0158 | 0.0158 |
| *5* | 0.0158 | 0.0158 | 0.0158 | 0.0158 |
| *6* | 0.0158 | 0.0158 | 0.0158 | 0.0158 |
| *7* | 0.0158 | 0.0158 | 0.0158 | 0.0158 |
| *8* | 0.0158 | 0.0158 | 0.0158 | 0.0158 |
| *9* | 0.0158 | 0.0158 | 0.0158 | 0.0158 |
| *10* | 0.0158 | 0.0158 | 0.0158 | 0.0158 |
| *11* | 0.0158 | 0.0158 | 0.0158 | 0.0158 |
| *12* | 0.0158 | 0.0158 | 0.0158 | 0.0158 |

*Notes:*

*The gross moment of inertia was calculated using half the width of the flange.*

*For details on the section dimensions, see section 5.12*

$I_{gross} = 0.4\ I_{eff}$

**Table A-11 Twelve storey building A, beam nominal moments at column face after redistribution**

| | *Beam nominal moments at column face after redistribution (kN m)* | | | |
|---|---|---|---|---|
| | *Model* | | | |
| *Level* | *12A/6/E/T* | *12A/6/3101/T* | *12A/6/1170/T* | *12A/6/E/B* |
| *1* | 673.2 | 673.2 | 673.2 | 673.2 |
| *2* | 699.0 | 699.0 | 699.0 | 699.0 |
| *3* | 647.6 | 647.6 | 647.6 | 647.6 |
| *4* | 588.3 | 588.3 | 588.3 | 588.3 |
| *5* | 528.5 | 528.5 | 528.5 | 528.5 |
| *6* | 528.5 | 528.5 | 528.5 | 528.5 |
| *7* | 528.5 | 528.5 | 528.5 | 528.5 |
| *8* | 528.5 | 528.5 | 528.5 | 528.5 |
| *9* | 528.5 | 528.5 | 528.5 | 528.5 |
| *10* | 528.5 | 528.5 | 528.5 | 528.5 |
| *11* | 528.5 | 528.5 | 528.5 | 528.5 |
| *12* | 528.5 | 528.5 | 528.5 | 528.5 |

*Notes:*

*Since these are moments after redistribution, they are along the three spans at each level.*

$Mn = M^{*}/\phi$ ; *where* $\phi = 0.85$

*The over-strength factor used for column design is 1.1.*

**Table A-12 Twelve storey building A, columns effective moment of inertia**

| | *Interior and exterior columns effective moment of inertia* $(m^4)$ | | | |
|---|---|---|---|---|
| | *Model* | | | |
| *Level* | *12A/6/E/T* | *12A/6/3101/T* | *12A/6/1170/T* | *12A/6/E/B* |
| *1* | 0.00914 | 0.00914 | 0.00914 | 0.00914 |
| *2* | 0.00914 | 0.00914 | 0.00914 | 0.00914 |
| *3* | 0.00914 | 0.00914 | 0.00914 | 0.00914 |
| *4* | 0.00914 | 0.00914 | 0.00914 | 0.00914 |
| *5* | 0.00914 | 0.00914 | 0.00914 | 0.00914 |
| *6* | 0.00914 | 0.00914 | 0.00914 | 0.00914 |
| *7* | 0.00914 | 0.00914 | 0.00914 | 0.00914 |
| *8* | 0.00914 | 0.00914 | 0.00914 | 0.00914 |
| *9* | 0.00914 | 0.00914 | 0.00914 | 0.00914 |
| *10* | 0.00914 | 0.00914 | 0.00914 | 0.00914 |
| *11* | 0.00914 | 0.00914 | 0.00914 | 0.00914 |
| *12* | 0.00914 | 0.00914 | 0.00914 | 0.00914 |

*Notes:*

*For details on the section dimensions, see section 5.12*

$I_{gross} = 0.4\ I_{eff}$

**Table A-13 Twelve storey building A, interior column nominal moments at beam face**

| | *Interior Column nominal moments at beam face (kN m)* | | | |
|---|---|---|---|---|
| | *Model* | | | |
| *Level* | *12A/6/E/T* | *12A/6/3101/T* | *12A/6/1170/T* | *12A/6/E/B* |
| *G.L.* | 753.0 | 753.0 | 753.0 | 753.0 |
| *1* | *Elastic columns* | 993.0 | 592.1 | *Elastic columns* |
| *2* | | 1059.0 | 617.3 | |
| *3* | | 1120.0 | 577.4 | |
| *4* | | 1148.0 | 530.2 | |
| *5* | | 1044.0 | 482.3 | |
| *6* | | 1065.0 | 488.5 | |
| *7* | | 1084.0 | 496.4 | |
| *8* | | 1109.0 | 507.1 | |
| *9* | | 1143.0 | 522.7 | |
| *10* | | 1205.0 | 548.2 | |
| *11* | | 892.0 | 579.6 | |
| *12* | | 120.0 | 507.7 | |

Notes:

At ground level, design moments come from Modal analysis plus P-delta induced forces factorised by a factor of 1 / 0.85.

**Table A-14 Twelve storey building A, exterior column nominal moments at beam face**

| | *Exterior Column nominal moments at beam face (kN m)* | | | |
|---|---|---|---|---|
| | *Model* | | | |
| *Level* | *12A/6/E/T* | *12A/6/3101/T* | *12A/6/1170/T* | *12A/6/E/B* |
| *G.L.* | 643.8 | 643.8 | 643.8 | 643.8 |
| *1* | *Elastic columns* | 478.0 | 236.8 | *Elastic columns* |
| *2* | | 562.0 | 246.9 | |
| *3* | | 621.1 | 230.9 | |
| *4* | | 617.5 | 212.1 | |
| *5* | | 566.2 | 192.9 | |
| *6* | | 582.0 | 195.4 | |
| *7* | | 597.0 | 198.6 | |
| *8* | | 617.7 | 202.8 | |
| *9* | | 649.0 | 209.1 | |
| *10* | | 697.1 | 219.3 | |
| *11* | | 582.8 | 231.8 | |
| *12* | | 72.0 | 203.1 | |

Notes:

At ground level, design moments come from Modal analysis plus P-delta induced forces factorised by a factor of 1 / 0.85.

**Table A-15 Twelve storey building A, interior column design axial forces**

| | *Interior Column design axial forces. (kN)** | | | |
|---|---|---|---|---|
| | *Model* | | | |
| *Level* | *12A/6/E/T* | *12A/6/3101/T* | *12A/6/1170/T* | *12A/6/E/B* |
| *G.L.* | -1479.2 | -1479.2 | -1479.2 | -1479.2 |
| *1* | *Elastic columns* | -1354.7 | -1354.7 | *Elastic columns* |
| *2* | | -1230.3 | -1230.3 | |
| *3* | | -1105.8 | -1105.8 | |
| *4* | | -981.4 | -981.4 | |
| *5* | | -856.9 | -856.9 | |
| *6* | | -732.4 | -732.4 | |
| *7* | | -608.0 | -608.0 | |
| *8* | | -483.5 | -483.5 | |
| *9* | | -359.1 | -359.1 | |
| *10* | | -234.6 | -234.6 | |
| *11* | | -110.1 | -110.1 | |
| *12* | | -110.1 | -110.1 | |
| ** Positive forces mean tension.* | | | | |

**Table A-16 Twelve storey building A, exterior column design axial forces**

| | *Exterior Column design axial forces. (kN)** | | | |
|---|---|---|---|---|
| | *Model* | | | |
| *Level* | *12A/6/E/T* | *12A/6/3101/T* | *12A/6/1170/T* | *12A/6/E/B* |
| *G.L.* | 1042.6 | 1042.6 | 1042.6 | 1042.6 |
| *1* | *Elastic columns* | 936.1 | 936.1 | *Elastic columns* |
| *2* | | 829.3 | 829.3 | |
| *3* | | 736.1 | 736.1 | |
| *4* | | 656.6 | 656.6 | |
| *5* | | 588.8 | 588.8 | |
| *6* | | 518.4 | 518.4 | |
| *7* | | 443.8 | 443.8 | |
| *8* | | 366.7 | 366.7 | |
| *9* | | 284.5 | 284.5 | |
| *10* | | 198.9 | 198.9 | |
| *11* | | 106.6 | 106.6 | |
| *12* | | 106.6 | 106.6 | |
| ** Positive forces mean tension.* | | | | |

**Table A-17 Twelve storey building A, interior column reinforcement ratios**

| | *Interior column reinforcement ratios* | | | |
|---|---|---|---|---|
| | *Model* | | | |
| *Level* | *12A/6/E/T* | *12A/6/3101/T* | *12A/6/1170/T* | *12A/6/E/B* |
| *G.L.* | 0.0080 | 0.0080 | 0.0080 | 0.0080 |
| *1* | *Elastic columns* | 0.0115 | 0.0031 | *Elastic columns* |
| *2* | | 0.0137 | 0.0043 | |
| *3* | | 0.0158 | 0.0043 | |
| *4* | | 0.0164 | 0.0033 | |
| *5* | | 0.0150 | 0.0031 | |
| *6* | | 0.0162 | 0.0041 | |
| *7* | | 0.0174 | 0.0051 | |
| *8* | | 0.0189 | 0.0062 | |
| *9* | | 0.0203 | 0.0074 | |
| *10* | | 0.0226 | 0.0086 | |
| *11* | | 0.0168 | 0.0103 | |
| *12* | | 0.008 | 0.0096 | |

Notes:

At ground level, design moments come from Modal analysis plus P-delta induced forces factorised by a factor of 1 / 0.85.

**Table A-18 Twelve storey building A, exterior column reinforcement ratios**

| | *Exterior column reinforcement ratios* | | | |
|---|---|---|---|---|
| | *Model* | | | |
| *Level* | *12A/6/E/T* | *12A/6/3101/T* | *12A/6/1170/T* | *12A/6/E/B* |
| *G.L.* | 0.0203 | 0.0203 | 0.0203 | 0.0203 |
| *1* | *Elastic columns* | 0.0160 | 0.0123 | *Elastic columns* |
| *2* | | 0.0178 | 0.0119 | |
| *3* | | 0.0183 | 0.0107 | |
| *4* | | 0.0176 | 0.0096 | |
| *5* | | 0.0160 | 0.0088 | |
| *6* | | 0.0158 | 0.0084 | |
| *7* | | 0.0158 | 0.0080 | |
| *8* | | 0.0156 | 0.0076 | |
| *9* | | 0.0158 | 0.0072 | |
| *10* | | 0.0162 | 0.0068 | |
| *11* | | 0.0131 | 0.0066 | |
| *12* | | 0.0080 | 0.0057 | |

Notes:

At ground level, design moments come from Modal analysis plus P-delta induced forces factorised by a factor of 1 / 0.85.

### *A.3 Twelve Storey Building B*

The post-yield stiffness *(r)* used for all the members in the twelve storey B structures is ***0.004***.

**Table A-19 Twelve storey building B, beams effective moment of inertia**

| | *Beams effective moment of inertia ($I_{eff}$) ($m^4$)* | | |
|---|---|---|---|
| | *Model* | | |
| *Level* | *12B/6/E/T* | *12B/6/3101/T* | *12B/6/1170/T* |
| *1* | 0.01580 | 0.01580 | 0.01580 |
| *2* | 0.01580 | 0.01580 | 0.01580 |
| *3* | 0.01580 | 0.01580 | 0.01580 |
| *4* | 0.01580 | 0.01580 | 0.01580 |
| *5* | 0.01580 | 0.01580 | 0.01580 |
| *6* | 0.01580 | 0.01580 | 0.01580 |
| *7* | 0.00929 | 0.00929 | 0.00929 |
| *8* | 0.00929 | 0.00929 | 0.00929 |
| *9* | 0.00929 | 0.00929 | 0.00929 |
| *10* | 0.00929 | 0.00929 | 0.00929 |
| *11* | 0.00929 | 0.00929 | 0.00929 |
| *12* | 0.00929 | 0.00929 | 0.00929 |

*Notes:*

*The gross moment of inertia was calculated using half the width of the flange.*

*For details on the section dimensions, see section 5.12*

$I_{gross} = 0.4\ I_{eff}$

**Table A-20 Twelve storey building B, beam nominal moments at column face after redistribution**

| | *Beam nominal moments at column face after redistribution (kN m)* | | |
|---|---|---|---|
| | *Model* | | |
| *Level* | *12B/6/E/T* | *12B/6/3101/T* | *12B/6/1170/T* |
| *1* | 632.0 | 632.0 | 632.0 |
| *2* | 667.6 | 667.6 | 667.6 |
| *3* | 619.4 | 619.4 | 619.4 |
| *4* | 561.5 | 561.5 | 561.5 |
| *5* | 509.4 | 509.4 | 509.4 |
| *6* | 509.4 | 509.4 | 509.4 |
| *7* | 406.9 | 406.9 | 406.9 |
| *8* | 369.9 | 369.9 | 369.9 |
| *9* | 369.9 | 369.9 | 369.9 |
| *10* | 369.9 | 369.9 | 369.9 |
| *11* | 369.9 | 369.9 | 369.9 |
| *12* | 369.9 | 369.9 | 369.9 |

*Notes:*

*Since these are moments after redistribution, they are along the three spans at each level.*

$Mn = M^{*}/\phi$ ; *where* $\phi = 0.85$

*The over-strength factor used for column design is 1.1.*

**Table A-21 Twelve storey building B, columns effective moment of inertia**

| | *Interior and exterior columns effective moment of inertia ($m^4$)* | | |
|---|---|---|---|
| | *Model* | | |
| *Level* | *12B/6/E/T* | *12B/6/3101/T* | *12B/6/1170/T* |
| *1* | 0.01010 | 0.01010 | 0.01010 |
| *2* | 0.01010 | 0.01010 | 0.01010 |
| *3* | 0.01010 | 0.01010 | 0.01010 |
| *4* | 0.01010 | 0.01010 | 0.01010 |
| *5* | 0.01010 | 0.01010 | 0.01010 |
| *6* | 0.01010 | 0.01010 | 0.01010 |
| *7* | 0.00743 | 0.00743 | 0.00743 |
| *8* | 0.00743 | 0.00743 | 0.00743 |
| *9* | 0.00743 | 0.00743 | 0.00743 |
| *10* | 0.00743 | 0.00743 | 0.00743 |
| *11* | 0.00743 | 0.00743 | 0.00743 |
| *12* | 0.00743 | 0.00743 | 0.00743 |

*Notes:*

*For details on the section dimensions, see section 5.12*

$I_{gross} = 0.4\ I_{eff}$

**Table A-22 Twelve storey building B, interior column nominal moments at beam face**

| | *Interior Column nominal moments at beam face (kN m)* | | |
|---|---|---|---|
| | *Model* | | |
| *Level* | *12B/6/E/T* | *12B/6/3101/T* | *12B/6/1170/T* |
| *G.L.* | 737.2 | 737.2 | 737.2 |
| *1* | *Elastic columns* | 942.0 | 558.2 |
| *2* | | 1014.0 | 594.8 |
| *3* | | 1074.0 | 559.9 |
| *4* | | 1102.0 | 516.1 |
| *5* | | 997.0 | 475.9 |
| *6* | | 992.0 / 966.0* | 482.7 / 482.7* |
| *7* | | 769.0 | 423.3 |
| *8* | | 790.7 | 395.9 |
| *9* | | 832.0 | 413.3 |
| *10* | | 885.0 | 442.5 |
| *11* | | 674.0 | 456.6 |
| *12* | | 129.0 | 377.5 |

Notes:

At ground level, design moments come from Modal analysis plus P-delta induced forces factorised by a factor of 1 / 0.85.

*At level 6, there's a change of the column section below and above the beam. Because of this, each section is designed with a different nominal moment. The values shown here are below / above.

**Table A-23 Twelve storey building B, exterior column nominal moments at beam face**

| | *Exterior Column nominal moments at beam face (kN m)* | | |
|---|---|---|---|
| | *Model* | | |
| *Level* | *12B/6/E/T* | *12B/6/3101/T* | *12B/6/1170/T* |
| *G.L.* | 632.0 | 632.0 | 632.0 |
| *1* | Elastic columns | 465.0 | 279.1 |
| *2* | | 540.3 | 297.4 |
| *3* | | 601.0 | 280.0 |
| *4* | | 595.0 | 258.0 |
| *5* | | 558.0 | 237.9 |
| *6* | | 478.0 / 570.6* | 241.4 |
| *7* | | 425.0 | 211.7 |
| *8* | | 442.0 | 197.9 |
| *9* | | 476.0 | 206.7 |
| *10* | | 515.0 | 221.2 |
| *11* | | 437.0 | 228.3 |
| *12* | | 65.0 | 188.7 |

Notes:

At ground level, design moments come from Modal analysis plus P-delta induced forces factorised by a factor of 1 / 0.85.

*At level 6, there's a change of the column section below and above the beam. Because of this, each section is designed with a different nominal moment. The values shown here are below / above.

**Table A-24 Twelve storey building B, interior column design axial forces**

| | *Interior Column design axial forces. (kN)** | | |
|---|---|---|---|
| | *Model* | | |
| *Level* | *12B/6/E/T* | *12B/6/3101/T* | *12B/6/1170/T* |
| *G.L.* | -1401.9 | -1401.9 | -1401.9 |
| *1* | Elastic columns | -1276.5 | -1276.5 |
| *2* | | -1151.1 | -1151.1 |
| *3* | | -1025.6 | -1025.6 |
| *4* | | -900.2 | -900.2 |
| *5* | | -774.8 | -774.8 |
| *6* | | -649.4 | -649.4 |
| *7* | | -538.8 | -538.8 |
| *8* | | -428.2 | -428.2 |
| *9* | | -317.6 | -317.6 |
| *10* | | -207.0 | -207.0 |
| *11* | | -96.4 | -96.4 |
| *12* | | -96.4 | -96.4 |

* *Positive forces mean tension.*

**Table A-25 Twelve storey building B, exterior column design axial forces**

| | *Exterior Column design axial forces. (kN)** | | |
|---|---|---|---|
| | *Model* | | |
| *Level* | *12B/6/E/T* | *12B/6/3101/T* | *12B/6/1170/T* |
| *G.L.* | 918.8 | 918.8 | 918.8 |
| *1* | *Elastic columns* | 806.5 | 806.5 |
| *2* | | 690.5 | 690.5 |
| *3* | | 588.5 | 588.5 |
| *4* | | 501.3 | 501.3 |
| *5* | | 424.9 | 424.9 |
| *6* | | 345.8 | 345.8 |
| *7* | | 288.0 | 288.0 |
| *8* | | 240.0 | 240.0 |
| *9* | | 188.1 | 188.1 |
| *10* | | 133.5 | 133.5 |
| *11* | | 73.9 | 73.9 |
| *12* | | 73.9 | 73.9 |
| ** Positive forces mean tension.* | | | |

**Table A-26 Twelve storey building B, interior column reinforcement ratios**

| | *Interior column reinforcement ratios* | | |
|---|---|---|---|
| | *Model* | | |
| *Level* | *12B/6/E/T* | *12B/6/3101/T* | *12B/6/1170/T* |
| *G.L.* | 0.0052 | 0.0052 | 0.0052 |
| *1* | *Elastic columns* | 0.0099 | 0.0022 |
| *2* | | 0.0121 | 0.0038 |
| *3* | | 0.0141 | 0.0038 |
| *4* | | 0.0145 | 0.0038 |
| *5* | | 0.0133 | 0.0036 |
| *6* | | 0.0167 / 0.0185* | 0.0058 / 0.0064* |
| *7* | | 0.0143 | 0.0057 |
| *8* | | 0.0149 | 0.0057 |
| *9* | | 0.0167 | 0.0068 |
| *10* | | 0.0189 | 0.0081 |
| *11* | | 0.0145 | 0.0099 |
| *12* | | 0.0022 | 0.0081 |

Notes:
At ground level, design moments come from Modal analysis plus P-delta induced forces factorised by a factor of 1 / 0.85.
*At level 6, there's a change of the column section below and above the beam. Both sections have the same amount of steel. The values shown here are reinforcement ratios below / above the beam.

**Table A-27 Twelve storey building B, exterior column reinforcement ratios**

| | *Exterior column reinforcement ratios* | | |
|---|---|---|---|
| | *Model* | | |
| *Level* | *12B/6/E/T* | *12B/6/3101/T* | *12B/6/1170/T* |
| *G.L.* | 0.0181 | 0.0181 | 0.0181 |
| *1* | *Elastic columns* | 0.0141 | 0.0105 |
| *2* | | 0.0157 | 0.0101 |
| *3* | | 0.0161 | 0.0091 |
| *4* | | 0.0153 | 0.0079 |
| *5* | | 0.0139 | 0.0071 |
| *6* | | 0.0145 / 0.0160 | 0.0071 / 0.0079* |
| *7* | | 0.0125 | 0.0068 |
| *8* | | 0.0125 | 0.0062 |
| *9* | | 0.0130 | 0.0059 |
| *10* | | 0.0134 | 0.0057 |
| *11* | | 0.0112 | 0.0057 |
| *12* | | 0.0020 | 0.0048 |

Notes:

At ground level, design moments come from Modal analysis plus P-delta induced forces factorised by a factor of 1 / 0.85.

*At level 6, there's a change of the column section below and above the beam. Both sections have the same amount of steel. The values shown here are reinforcement ratios below / above the beam.

## *A.4 Four Storey Building*

The post-yield stiffness *(r)* used for all the members in the four storey structures is ***0.007***.

**Table A-28 Four storey structure beams effective moment of inertia**

| | *Beams effective moment of inertia ($I_{eff}$) ($m^4$)* | | |
|---|---|---|---|
| | *Model* | | |
| *Level* | *4/6/E/T* | *4/6/3101/T* | *4/6/1170/T* |
| *1* | 0.00585 | 0.00585 | 0.00585 |
| *2* | 0.00585 | 0.00585 | 0.00585 |
| *3* | 0.00585 | 0.00585 | 0.00585 |
| *4* | 0.00585 | 0.00585 | 0.00585 |

*Notes:*

*The gross moment of inertia was calculated using half the width of the flange.*

*For details on the section dimensions, see section 5.12*

$I_{gross} = 0.4\ I_{eff}$

**Table A-29 Four storey structure beam nominal moments at column face after redistribution**

| | *Beam nominal moments at column face after redistribution (kN m)* | | |
|---|---|---|---|
| | *Model* | | |
| *Level* | *4/6/E/T* | *4/6/3101/T* | *4/6/1170/T* |
| *1* | 345.5 | 345.5 | 345.5 |
| *2* | 303.8 | 303.8 | 303.8 |
| *3* | 215.0 | 215.0 | 215.0 |
| *4* | 215.0 | 215.0 | 215.0 |

*Notes:*

*Since these are moments after redistribution, they are along the three spans at each level.*

$Mn = M^* / \phi$ ; *where* $\phi = 0.85$

*The over-strength factor used for column design is 1.1.*

**Table A-30 Four storey interior and exterior columns effective moment of inertia**

| | *Interior and exterior columns effective moment of inertia ($m^4$)* | | |
|---|---|---|---|
| | *Model* | | |
| *Level* | *4/6/E/T* | *4/6/3101/T* | *4/6/1170/T* |
| *1* | 0.00407 | 0.00407 | 0.00407 |
| *2* | 0.00407 | 0.00407 | 0.00407 |
| *3* | 0.00407 | 0.00407 | 0.00407 |
| *4* | 0.00407 | 0.00407 | 0.00407 |

*Notes:*

*For details on the section dimensions, see section 5.12*

$I_{gross} = 0.4\ I_{eff}$

**Table A-31 Four storey interior column nominal moments at beam face**

| | *Interior Column nominal moments at beam face (kN m)* | | |
|---|---|---|---|
| | *Model* | | |
| *Level* | *4/6/E/T* | *4/6/3101/T* | *4/6/1170/T* |
| *G.L.* | 448.0 | 448.0 | 448.0 |
| *1* | *Elastic columns* | 505.8 | 345.0 |
| *2* | | 879.0 | 326.9 |
| *3* | | 339.0 | 250.3 |
| *4* | | 165.0 | 171.9 |

Notes:

At ground level, design moments come from Modal analysis plus P-delta induced forces factorised by a factor of 1 / 0.85.

**Table A-32 Four storey exterior column nominal moments at beam face**

| | *Exterior Column nominal moments at beam face (kN m)* | | |
|---|---|---|---|
| | *Model* | | |
| *Level* | *4/6/E/T* | *4/6/3101/T* | *4/6/1170/T* |
| *G.L.* | 389.0 | 389.0 | 389.0 |
| *1* | *Elastic columns* | 257.0 | 172.5 |
| *2* | | 648.0 | 163.4 |
| *3* | | 206.0 | 125.1 |
| *4* | | 100.0 | 86.0 |

**Table A-33 Four storey interior column design axial forces**

| | *Interior Column design axial forces. (kN)** | | |
|---|---|---|---|
| | *Model* | | |
| *Level* | *4/6/E/T* | *4/6/3101/T* | *4/6/1170/T* |
| *G.L.* | -311.3 | -311.3 | -311.3 |
| *1* | *Elastic columns* | -231.0 | -231.0 |
| *2* | | -150.7 | -150.7 |
| *3* | | -70.4 | -70.4 |
| *4* | | -70.4 | -70.4 |
| ** Positive forces mean tension.* | | | |

**Table A-34 Four storey exterior column design axial forces**

| | *Exterior Column design axial forces. (kN)* | | |
|---|---|---|---|
| | *Model* | | |
| *Level* | *4/6/E/T* | *4/6/3101/T* | *4/6/1170/T* |
| *G.L.* | 138.7 | 138.7 | 138.7 |
| *1* | *Elastic columns* | 85.2 | 85.2 |
| *2* | | 42.8 | 42.8 |
| *3* | | 26.3 | 26.3 |
| *4* | | 26.3 | 26.3 |
| ** Positive forces mean tension.* | | | |

**Table A-35 Four storey interior column reinforcement ratios**

| | *Interior Column reinforcement ratios.* | | |
|---|---|---|---|
| | *Model* | | |
| *Level* | *4/6/E/T* | *4/6/3101/T* | *4/6/1170/T* |
| *G.L.* | 0.0144 | 0.0144 | 0.0144 |
| *1* | *Elastic columns* | 0.0176 | 0.0109 |
| *2* | | 0.0326 | 0.0112 |
| *3* | | 0.0115 | 0.0090 |
| *4* | | 0.0054 | 0.0058 |

**Table A-36 Four storey exterior column reinforcement ratios**

| | *Exterior Column reinforcement ratios.* | | |
|---|---|---|---|
| | *Model* | | |
| *Level* | *4/6/E/T* | *4/6/3101/T* | *4/6/1170/T* |
| *G.L.* | 0.0166 | 0.0166 | 0.0166 |
| *1* | *Elastic columns* | 0.0115 | 0.0074 |
| *2* | | 0.0266 | 0.0067 |
| *3* | | 0.0083 | 0.0048 |
| *4* | | 0.0038 | 0.0035 |

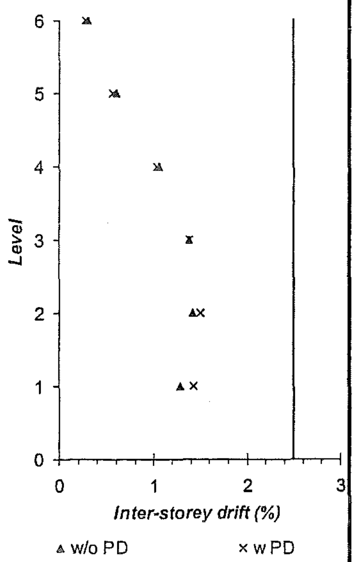
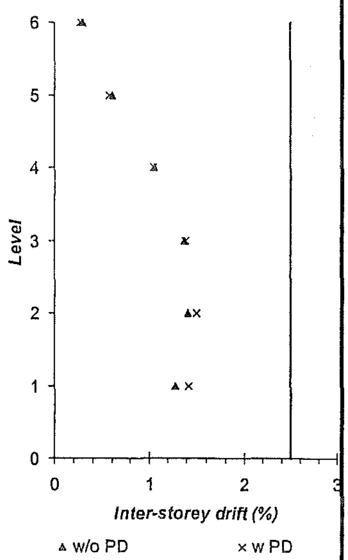
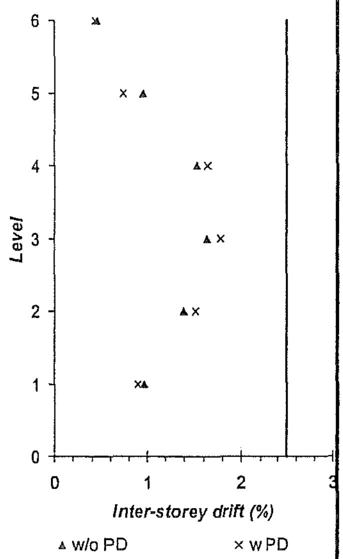
## APPENDIX B. NON-LINEAR TIME-HISTORY ANALYSES RESULTS

The results given in this appendix for each structure for each of the ground acceleration records are:

1. The maximum base shear with and without P-delta effects
2. A graph and the values of the maximum inter-storey drifts with and without P-delta effects
3. A graph showing the maximum displacements with and without P-delta effects
4. The maximum rotation of plastic hinge on the left side of the beams in the central span
5. The maximum rotation of the plastic hinge on one of the interior columns for levels one through six.

### *B.1 Six Storey Building*

**Table B-1 Result comparison between models 6/6/E/T, 6/6/3101/T and 6/6/1170/T for Modified El Centro NS ground acceleration record**

| | *Modified El Centro NS* | | | | | |
|---|---|---|---|---|---|---|
| | *6/6/E/T* | | *6/6/3101/T* | | *6/6/1170/T* | |
| | *Maximum base shear (kN)* | | *Maximum base shear (kN)* | | *Maximum base shear (kN)* | |
| *W/o P-Δ* | 1348.1 | | 1346.0 | | 1252.9 | |
| *With P-Δ* | 1493.3 | | 1492.2 | | 1353.1 | |
| *Inter-storey drifts* |  | |  | |  | |
| | *Inter-storey drifts envelope (% of storey height)* | | *Inter-storey drifts envelope (% of storey height)* | | *Inter-storey drifts envelope (% of storey height)* | |
| *Storey* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* |
| *1* | 1.28 | 1.42 | 1.28 | 1.42 | 0.96 | 0.89 |
| *2* | 1.42 | 1.50 | 1.41 | 1.50 | 1.38 | 1.51 |
| *3* | 1.38 | 1.38 | 1.37 | 1.38 | 1.64 | 1.78 |
| *4* | 1.06 | 1.03 | 1.06 | 1.04 | 1.52 | 1.64 |
| *5* | 0.59 | 0.56 | 0.59 | 0.56 | 0.95 | 0.73 |
| *6* | 0.30 | 0.28 | 0.30 | 0.28 | 0.45 | 0.43 |

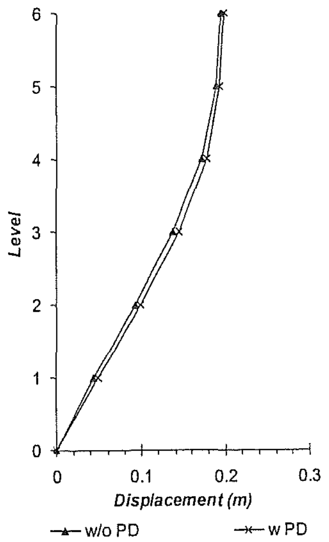
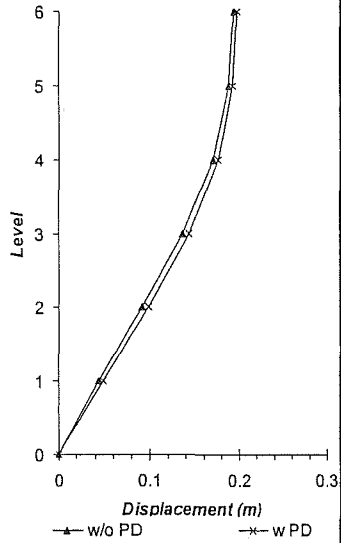
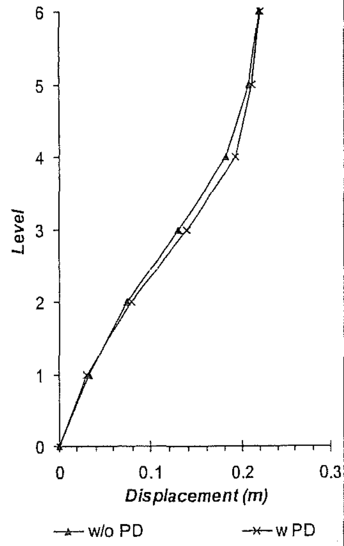
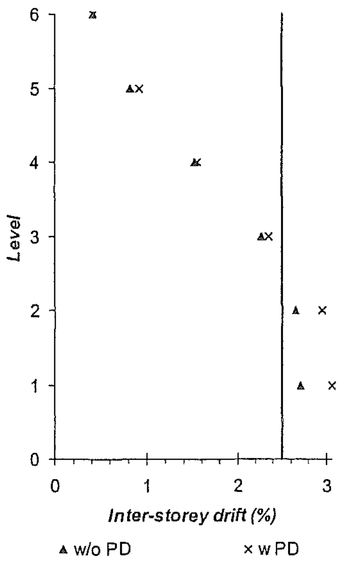
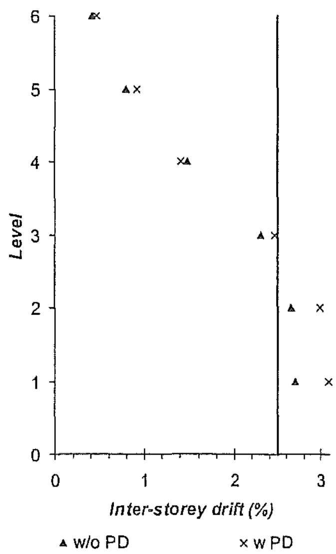
| Modified El Centro NS | | | | | | |
|---|---|---|---|---|---|---|
| | 6/6/E/T | | 6/6/3101/T | | 6/6/1170/T | |
| *Maximum displacements* |   | |   | |   | |
| | ***Maximum Displacements (m)*** | | ***Maximum Displacements (m)*** | | ***Maximum Displacements (m)*** | |
| *Level* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* |
| *1* | 0.044 | 0.048 | 0.044 | 0.048 | 0.033 | 0.030 |
| *2* | 0.092 | 0.099 | 0.092 | 0.099 | 0.074 | 0.079 |
| *3* | 0.137 | 0.144 | 0.136 | 0.144 | 0.130 | 0.139 |
| *4* | 0.171 | 0.177 | 0.171 | 0.177 | 0.181 | 0.192 |
| *5* | 0.189 | 0.193 | 0.189 | 0.193 | 0.208 | 0.212 |
| *6* | 0.194 | 0.198 | 0.194 | 0.197 | 0.218 | 0.220 |
| *Level* | ***Central span beam rotation (rad)*** | | ***Centre span beam rotation (rad)*** | | ***Centre span beam rotation (rad)*** | |
| *1* | 0.0128 | | 0.0127 | | 0.0070 | |
| *2* | 0.0132 | | 0.0132 | | 0.0143 | |
| *3* | 0.0108 | | 0.0109 | | 0.0153 | |
| *4* | 0.0065 | | 0.0065 | | 0.0064 | |
| *5* | 0.0017 | | 0.0017 | | 0.0005 | |
| *6* | 0.0003 | | 0.0003 | | 0.0001 | |
| | ***Interior column rotation (rad)*** | | ***Interior column rotation (rad)*** | | ***Interior column rotation (rad)*** | |
| ***G. L.*** | 0.0122 | | 0.0122 | | 0.0071 | |
| ***1 Below*** | | | 0.0009 | | 0.0009 | |
| ***1 Above*** | | | 0.0015 | | 0.0072 | |
| ***2 Below*** | | | 0.0013 | | 0.0023 | |
| ***2 Above*** | | | 0.0013 | | 0.0048 | |
| ***3 Below*** | | | 0.0013 | | 0.0039 | |
| ***3 Above*** | *Elastic columns at upper levels* | | 0.0010 | | 0.0028 | |
| ***4 Below*** | | | 0.0014 | | 0.0104 | |
| ***4 Above*** | | | 0.0009 | | 0.0037 | |
| ***5 Below*** | | | 0.0012 | | 0.0065 | |
| ***5 Above*** | | | 0.0005 | | 0.0035 | |
| ***6 Below*** | | | 0.0021 | | 0.0052 | |

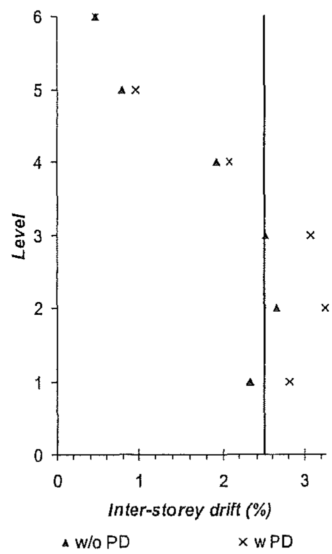
**Table B-2 Result comparison between models 6/6/E/T, 6/6/3101/T and 6/6/1170/T for Modified El Centro EW ground acceleration record**

| | *Modified El Centro EW* | | |
|---|---|---|---|
| | *6/6/E/T* | *6/6/3101/T* | *6/6/1170/T* |
| | *Maximum base shear (kN)* | *Maximum base shear (kN)* | *Maximum base shear (kN)* |
| *W/o P-Δ* | 1521.7 | 1511.8 | 1358.0 |
| *With P-Δ* | 1934.8 | 1907.0 | 1657.3 |

*Inter-storey drifts*




| | *Inter-storey drifts envelope (% of storey height)* | | *Inter-storey drifts envelope (% of storey height)* | | *Inter-storey drifts envelope (% of storey height)* | |
|---|---|---|---|---|---|---|
| *Storey* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* |
| *1* | 2.71 | 3.06 | 2.70 | 3.09 | 2.32 | 2.81 |
| *2* | 2.65 | 2.94 | 2.65 | 2.99 | 2.66 | 3.24 |
| *3* | 2.26 | 2.35 | 2.30 | 2.46 | 2.51 | 3.06 |
| *4* | 1.53 | 1.54 | 1.49 | 1.41 | 1.93 | 2.08 |
| *5* | 0.82 | 0.91 | 0.78 | 0.91 | 0.78 | 0.95 |
| *6* | 0.41 | 0.41 | 0.41 | 0.46 | 0.46 | 0.46 |

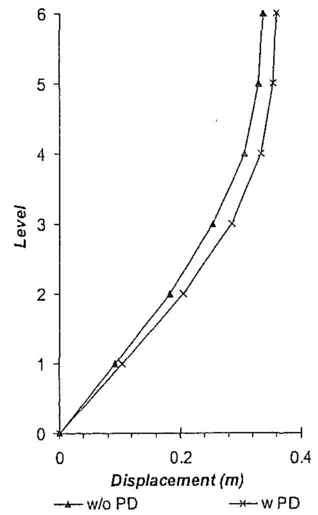
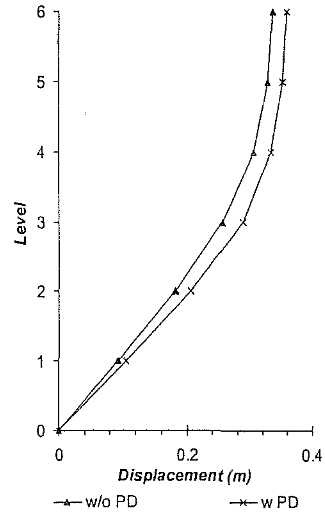
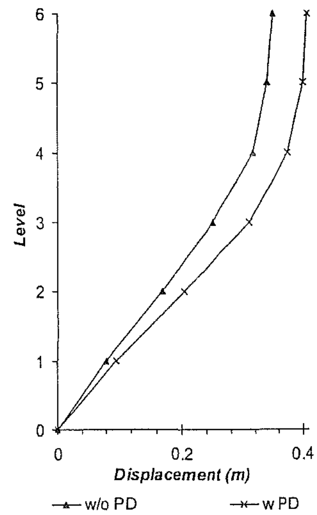
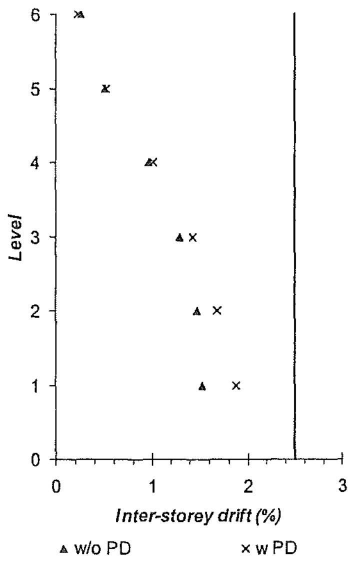
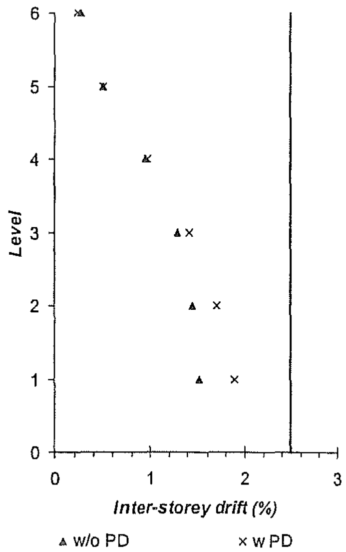
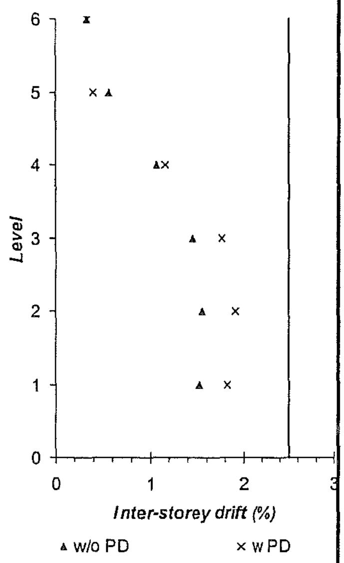
| Modified El Centro EW | | | | | | |
|---|---|---|---|---|---|---|
| | *6/6/E/T* | | *6/6/3101/T* | | *6/6/1170/T* | |
| *Maximum displacements* |  | |  | |  | |
| | *Maximum Displacements (m)* | | *Maximum Displacements (m)* | | *Maximum Displacements (m)* | |
| *Level* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* |
| *1* | 0.092 | 0.104 | 0.092 | 0.105 | 0.079 | 0.096 |
| *2* | 0.182 | 0.204 | 0.182 | 0.205 | 0.169 | 0.205 |
| *3* | 0.254 | 0.284 | 0.256 | 0.289 | 0.250 | 0.308 |
| *4* | 0.306 | 0.333 | 0.306 | 0.333 | 0.315 | 0.375 |
| *5* | 0.330 | 0.354 | 0.329 | 0.353 | 0.339 | 0.399 |
| *6* | 0.336 | 0.359 | 0.337 | 0.360 | 0.348 | 0.406 |
| *Level* | *Central span beam rotation (rad)* | | *Mid-span beam rotation* (rad) | | *Mid-span beam rotation* (rad) | |
| *1* | 0.0300 | | 0.0300 | | 0.0248 | |
| *2* | 0.0263 | | 0.0273 | | 0.0261 | |
| *3* | 0.0185 | | 0.0164 | | 0.0186 | |
| *4* | 0.0105 | | 0.0101 | | 0.0041 | |
| *5* | 0.0047 | | 0.0048 | | 0.0005 | |
| *6* | 0.0004 | | 0.0003 | | 0.0001 | |
| | *Interior column rotation (rad)* | | *Interior column rotation (rad)* | | *Interior column rotation (rad)* | |
| *G. L.* | 0.0278 | | 0.0284 | | 0.0261 | |
| *1 Below* | | | 0.0013 | | 0.0035 | |
| *1 Above* | | | 0.0012 | | 0.0098 | |
| *2 Below* | | | 0.0020 | | 0.0087 | |
| *2 Above* | | | 0.0012 | | 0.0076 | |
| *3 Below* | | | 0.0065 | | 0.0141 | |
| *3 Above* | *Elastic columns at upper levels* | | 0.0010 | | 0.0059 | |
| *4 Below* | | | 0.0017 | | 0.0153 | |
| *4 Above* | | | 0.0008 | | 0.0042 | |
| *5 Below* | | | 0.0013 | | 0.0095 | |
| *5 Above* | | | 0.0006 | | 0.0039 | |
| *6 Below* | | | 0.0035 | | 0.0054 | |

**Table B-3 Result comparison between models 6/6/E/T, 6/6/3101/T and 6/6/1170/T for Modified TAFT1 ground acceleration record.**

| | *Modified TAFT1* | | | | | |
|---|---|---|---|---|---|---|
| | *6/6/E/T* | | *6/6/3101/T* | | *6/6/1170/T* | |
| | *Maximum base shear (kN)* | | *Maximum base shear (kN)* | | *Maximum base shear (kN)* | |
| *W/o P-Δ* | 1565.4 | | 1541.9 | | 1393.6 | |
| *With P-Δ* | 1712.7 | | 1714.0 | | 1559.3 | |
| *Inter-storey drifts* |  | |  | |  | |
| | *Inter-storey drifts envelope (% of storey height)* | | *Inter-storey drifts envelope (% of storey height)* | | *Inter-storey drifts envelope (% of storey height)* | |
| *Storey* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* |
| *1* | 1.52 | 1.88 | 1.52 | 1.90 | 1.52 | 1.82 |
| *2* | 1.48 | 1.69 | 1.45 | 1.71 | 1.56 | 1.93 |
| *3* | 1.28 | 1.43 | 1.29 | 1.41 | 1.46 | 1.77 |
| *4* | 0.96 | 1.01 | 0.95 | 0.97 | 1.07 | 1.16 |
| *5* | 0.50 | 0.52 | 0.51 | 0.51 | 0.57 | 0.40 |
| *6* | 0.26 | 0.22 | 0.27 | 0.23 | 0.33 | 0.32 |

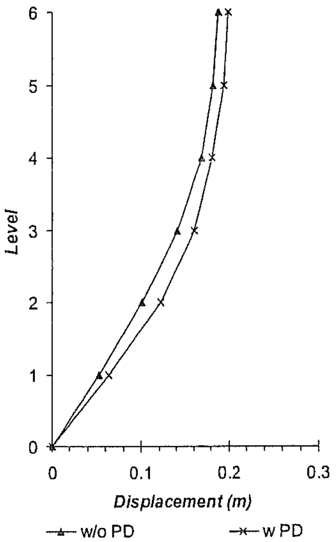
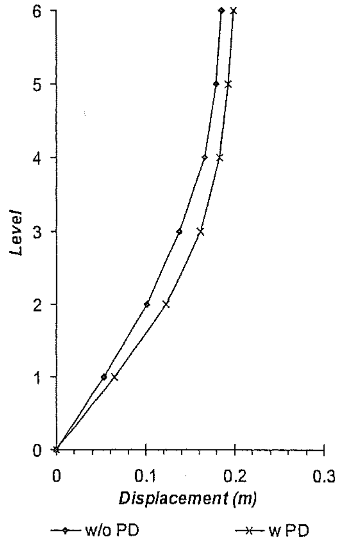
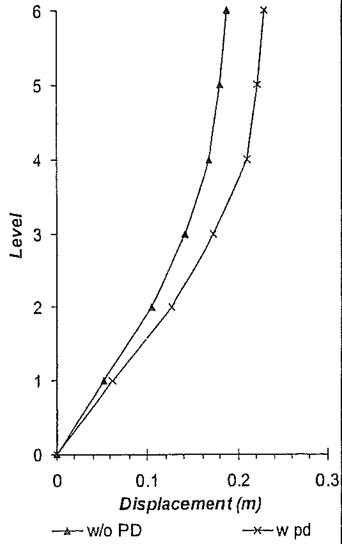
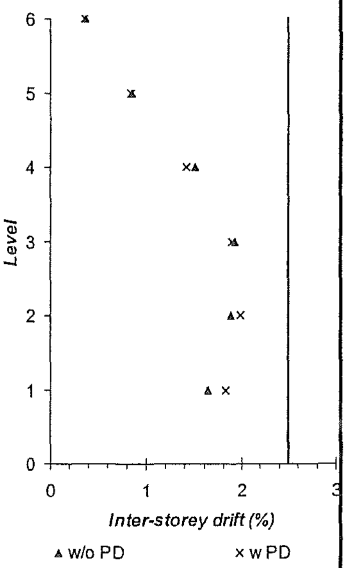
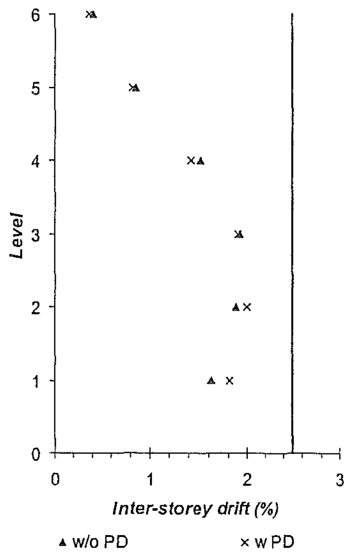
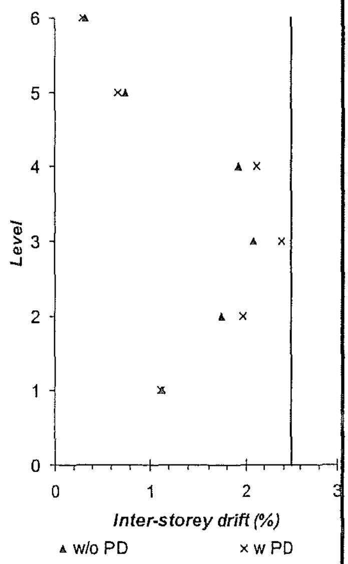
| | Modified TAFT1 | | | | | |
|---|---|---|---|---|---|---|
| | 6/6/E/T | | 6/6/3101/T | | 6/6/1170/T | |
| *Maximum displacements* |  | |  | |  | |
| | *Maximum Displacements (m)* | | *Maximum Displacements (m)* | | *Maximum Displacements (m)* | |
| *Level* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* |
| *1* | 0.052 | 0.064 | 0.052 | 0.064 | 0.052 | 0.062 |
| *2* | 0.101 | 0.121 | 0.101 | 0.123 | 0.105 | 0.126 |
| *3* | 0.141 | 0.161 | 0.138 | 0.162 | 0.141 | 0.173 |
| *4* | 0.169 | 0.181 | 0.166 | 0.183 | 0.168 | 0.211 |
| *5* | 0.183 | 0.194 | 0.180 | 0.193 | 0.180 | 0.221 |
| *6* | 0.188 | 0.200 | 0.185 | 0.199 | 0.187 | 0.228 |
| *Level* | *Central span beam rotation (rad)* | | *Central span beam rotation (rad)* | | *Central span beam rotation (rad)* | |
| *1* | 0.0169 | | 0.0171 | | 0.0169 | |
| *2* | 0.0136 | | 0.0139 | | 0.0153 | |
| *3* | 0.0111 | | 0.0106 | | 0.0113 | |
| *4* | 0.0055 | | 0.0054 | | 0.0023 | |
| *5* | 0.0011 | | 0.0011 | | 0.0005 | |
| *6* | 0.0003 | | 0.0003 | | 0.0001 | |
| | *Interior column rotation (rad)* | | *Interior column rotation (rad)* | | *Interior column rotation (rad)* | |
| *G. L.* | 0.0166 | | 0.0168 | | 0.0158 | |
| *1 Below* | | | 0.0012 | | 0.0010 | |
| *1 Above* | | | 0.0013 | | 0.0049 | |
| *2 Below* | | | 0.0017 | | 0.0070 | |
| *2 Above* | | | 0.0011 | | 0.0022 | |
| *3 Below* | | | 0.0015 | | 0.0069 | |
| *3 Above* | *Elastic columns at upper levels* | | 0.0009 | | 0.0018 | |
| *4 Below* | | | 0.0014 | | 0.0088 | |
| *4 Above* | | | 0.0006 | | 0.0023 | |
| *5 Below* | | | 0.0012 | | 0.0030 | |
| *5 Above* | | | 0.0005 | | 0.0024 | |
| *6 Below* | | | 0.0008 | | 0.0041 | |

**Table B-4 Result comparison between models 6/6/E/T, 6/6/3101/T and 6/6/1170/T for Modified TAFT2 ground acceleration record.**

| *Modified TAFT2* | | | | | | |
|---|---|---|---|---|---|---|
| | *6/6/E/T* | | *6/6/3101/T* | | *6/6/1170/T* | |
| | *Maximum base shear (kN)* | | *Maximum base shear (kN)* | | *Maximum base shear (kN)* | |
| *W/o P-Δ* | 1429.3 | | 1435.4 | | 1349.9 | |
| *With P-Δ* | 1537.5 | | 1532.1 | | 1416.9 | |
| *Inter-storey drifts* |  | |  | |  | |
| | *Inter-storey drifts envelope (% of storey height)* | | *Inter-storey drifts envelope (% of storey height)* | | *Inter-storey drifts envelope (% of storey height)* | |
| *Storey* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* |
| *1* | 1.65 | 1.83 | 1.63 | 1.81 | 1.13 | 1.12 |
| *2* | 1.88 | 1.99 | 1.89 | 2.01 | 1.75 | 1.96 |
| *3* | 1.93 | 1.90 | 1.94 | 1.91 | 2.08 | 2.38 |
| *4* | 1.51 | 1.41 | 1.52 | 1.42 | 1.93 | 2.13 |
| *5* | 0.86 | 0.84 | 0.86 | 0.82 | 0.75 | 0.67 |
| *6* | 0.38 | 0.36 | 0.41 | 0.37 | 0.33 | 0.29 |

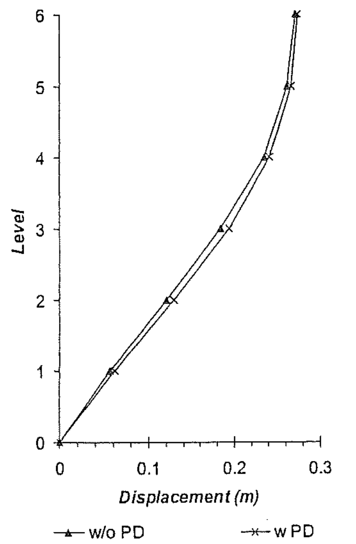
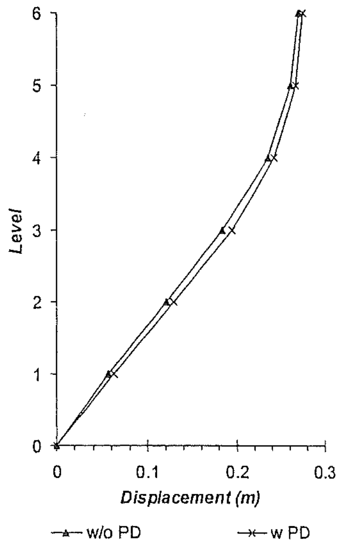
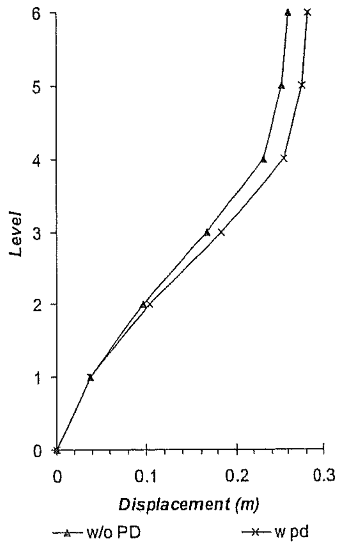
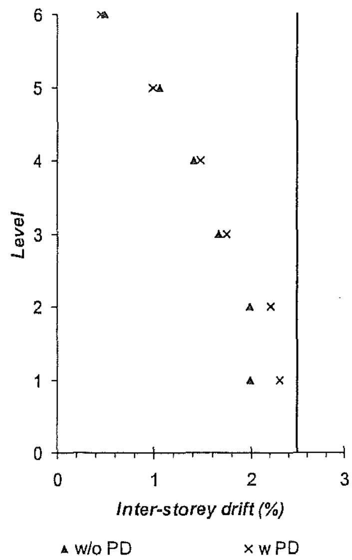
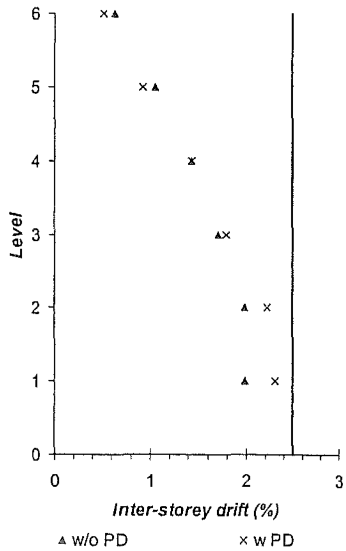
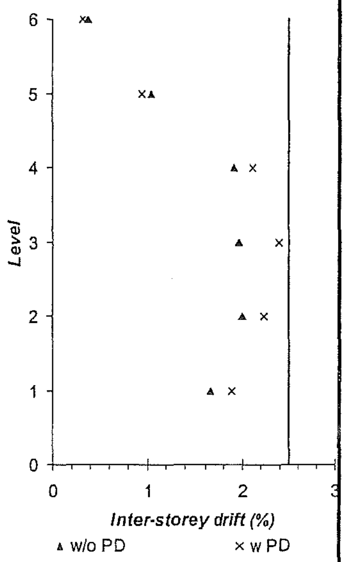
| Modified TAFT2 | | | | | | |
|---|---|---|---|---|---|---|
| | 6/6/E/T | | 6/6/3101/T | | 6/6/1170/T | |
| *Maximum displacements* |   | |   | |   | |
| | *Maximum Displacements (m)* | | *Maximum Displacements (m)* | | *Maximum Displacements (m)* | |
| *Level* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* |
| *1* | 0.056 | 0.062 | 0.055 | 0.062 | 0.038 | 0.038 |
| *2* | 0.120 | 0.129 | 0.120 | 0.129 | 0.096 | 0.103 |
| *3* | 0.184 | 0.193 | 0.184 | 0.194 | 0.166 | 0.183 |
| *4* | 0.235 | 0.241 | 0.236 | 0.243 | 0.231 | 0.255 |
| *5* | 0.261 | 0.266 | 0.261 | 0.267 | 0.253 | 0.276 |
| *6* | 0.271 | 0.274 | 0.271 | 0.276 | 0.260 | 0.282 |
| *Level* | *Central span beam rotation (rad)* | | *Central span beam rotation (rad)* | | *Central span beam rotation (rad)* | |
| *1* | 0.0175 | | 0.0176 | | 0.0096 | |
| *2* | 0.0188 | | 0.0189 | | 0.0185 | |
| *3* | 0.0162 | | 0.0164 | | 0.0205 | |
| *4* | 0.0093 | | 0.0091 | | 0.0044 | |
| *5* | 0.0036 | | 0.0034 | | 0.0005 | |
| *6* | 0.0003 | | 0.0003 | | 0.0001 | |
| | *Interior column rotation (rad)* | | *Interior column rotation (rad)* | | *Interior column rotation (rad)* | |
| *G. L.* | 0.0162 | | 0.0160 | | 0.0096 | |
| *1 Below* | | | 0.0011 | | 0.0010 | |
| *1 Above* | | | 0.0015 | | 0.0098 | |
| *2 Below* | | | 0.0013 | | 0.0025 | |
| *2 Above* | | | 0.0010 | | 0.0056 | |
| *3 Below* | | | 0.0014 | | 0.0046 | |
| *3 Above* | *Elastic columns at upper levels* | | 0.0008 | | 0.0032 | |
| *4 Below* | | | 0.0018 | | 0.0176 | |
| *4 Above* | | | 0.0007 | | 0.0017 | |
| *5 Below* | | | 0.0014 | | 0.0060 | |
| *5 Above* | | | 0.0005 | | 0.0015 | |
| *6 Below* | | | 0.0017 | | 0.0036 | |

**Table B-5 Result comparison between models 6/6/E/T, 6/6/3101/T and 6/6/1170/T for El Centro 1 ground acceleration record**

| *El Centro 1* | | | | | | |
|---|---|---|---|---|---|---|
| | *6/6/E/T* | | *6/6/3101/T* | | *6/6/1170/T* | |
| | *Maximum base shear (kN)* | | *Maximum base shear (kN)* | | *Maximum base shear (kN)* | |
| *W/o P-Δ* | 1436.8 | | 1437.3 | | 1353.8 | |
| *With P-Δ* | 1742.6 | | 1744.2 | | 1520.6 | |
| *Inter-storey drifts* |  | |  | |  | |
| | *Inter-storey drifts envelope (% of storey height)* | | *Inter-storey drifts envelope (% of storey height)* | | *Inter-storey drifts envelope (% of storey height)* | |
| *Storey* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* |
| *1* | 1.99 | 2.31 | 1.99 | 2.31 | 1.66 | 1.90 |
| *2* | 1.99 | 2.22 | 1.99 | 2.23 | 2.01 | 2.24 |
| *3* | 1.67 | 1.74 | 1.71 | 1.79 | 1.97 | 2.39 |
| *4* | 1.41 | 1.47 | 1.43 | 1.44 | 1.92 | 2.11 |
| *5* | 1.06 | 0.98 | 1.05 | 0.93 | 1.05 | 0.94 |
| *6* | 0.49 | 0.45 | 0.63 | 0.51 | 0.37 | 0.31 |

**El Centro 1**

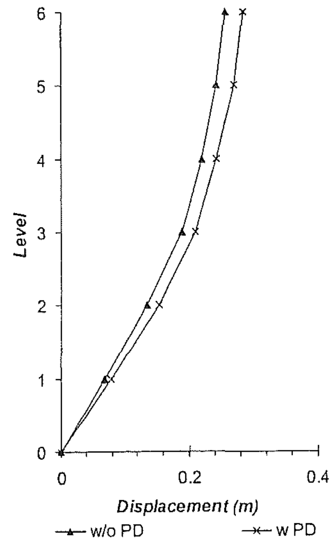
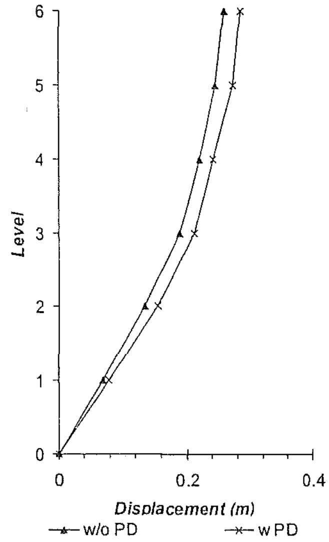
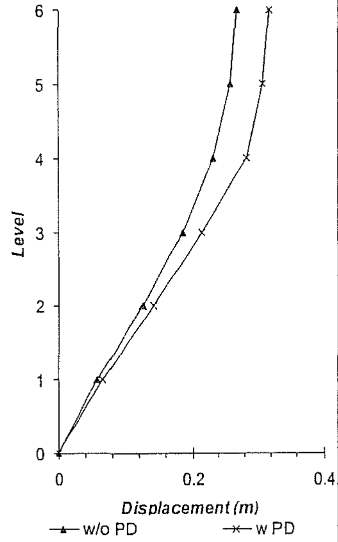
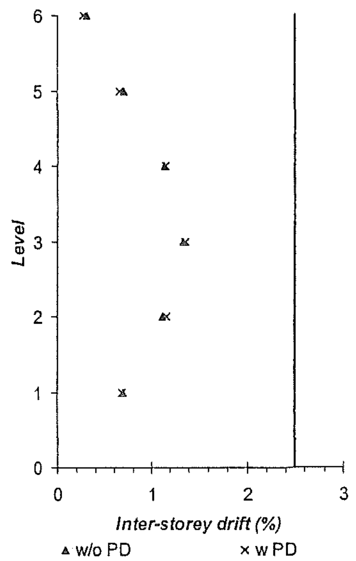
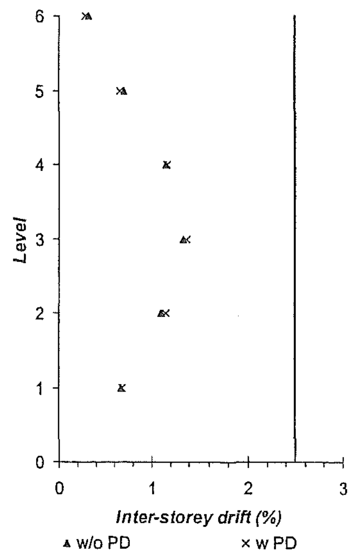
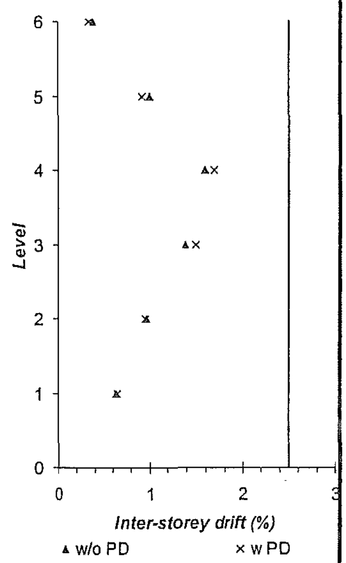
| | 6/6/E/T | | 6/6/3101/T | | 6/6/1170/T | |
|---|---|---|---|---|---|---|
| *Maximum displacements* |  | |  | |  | |
| | *Maximum Displacements (m)* | | *Maximum Displacements (m)* | | *Maximum Displacements (m)* | |
| *Level* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* |
| *1* | 0.068 | 0.079 | 0.068 | 0.078 | 0.056 | 0.065 |
| *2* | 0.135 | 0.154 | 0.135 | 0.154 | 0.125 | 0.141 |
| *3* | 0.188 | 0.210 | 0.189 | 0.210 | 0.185 | 0.213 |
| *4* | 0.220 | 0.242 | 0.220 | 0.242 | 0.231 | 0.283 |
| *5* | 0.242 | 0.270 | 0.245 | 0.272 | 0.259 | 0.308 |
| *6* | 0.255 | 0.283 | 0.259 | 0.284 | 0.269 | 0.318 |
| *Level* | ***Central span beam rotation (rad)*** | | ***Central span beam rotation (rad)*** | | ***Central span beam rotation (rad)*** | |
| *1* | 0.0220 | | 0.0219 | | 0.0178 | |
| *2* | 0.0185 | | 0.0188 | | 0.0194 | |
| *3* | 0.0153 | | 0.0148 | | 0.0182 | |
| *4* | 0.0110 | | 0.0107 | | 0.0061 | |
| *5* | 0.0055 | | 0.0049 | | 0.0005 | |
| *6* | 0.0004 | | 0.0003 | | 0.0001 | |
| | ***Interior column rotation (rad)*** | | ***Interior column rotation (rad)*** | | ***Interior column rotation (rad)*** | |
| ***G. L.*** | 0.0208 | | 0.0207 | | 0.0171 | |
| ***1 Below*** | ***Elastic columns at upper levels*** | | 0.0011 | | 0.0011 | |
| ***1 Above*** | | | 0.0013 | | 0.0055 | |
| ***2 Below*** | | | 0.0017 | | 0.0045 | |
| ***2 Above*** | | | 0.0010 | | 0.0066 | |
| ***3 Below*** | | | 0.0028 | | 0.0070 | |
| ***3 Above*** | | | 0.0009 | | 0.0044 | |
| ***4 Below*** | | | 0.0016 | | 0.0158 | |
| ***4 Above*** | | | 0.0007 | | 0.0034 | |
| ***5 Below*** | | | 0.0016 | | 0.0093 | |
| ***5 Above*** | | | 0.0005 | | 0.0015 | |
| ***6 Below*** | | | 0.0042 | | 0.0036 | |

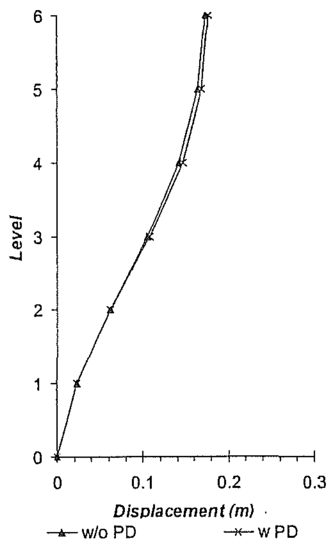
**Table B-6 Result comparison between models 6/6/E/T, 6/6/3101/T and 6/6/1170/T for El Centro 2 ground acceleration record**

| *El Centro 2* | | | | | | |
|---|---|---|---|---|---|---|
| | *6/6/E/T* | | *6/6/3101/T* | | *6/6/1170/T* | |
| | *Maximum base shear (kN)* | | *Maximum base shear (kN)* | | *Maximum base shear (kN)* | |
| *W/o P-Δ* | 1306.8 | | 1292.8 | | 1279.8 | |
| *With P-Δ* | 1360.2 | | 1360.1 | | 1347.8 | |
| *Inter-storey drifts* |  | |  | |  | |
| | *Inter-storey drifts envelope (% of storey height)* | | *Inter-storey drifts envelope (% of storey height)* | | *Inter-storey drifts envelope (% of storey height)* | |
| *Storey* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* |
| *1* | 0.69 | 0.67 | 0.67 | 0.68 | 0.63 | 0.64 |
| *2* | 1.12 | 1.16 | 1.10 | 1.15 | 0.97 | 0.94 |
| *3* | 1.34 | 1.35 | 1.33 | 1.37 | 1.38 | 1.51 |
| *4* | 1.14 | 1.14 | 1.15 | 1.16 | 1.59 | 1.69 |
| *5* | 0.70 | 0.66 | 0.70 | 0.66 | 0.99 | 0.91 |
| *6* | 0.31 | 0.27 | 0.31 | 0.28 | 0.37 | 0.33 |

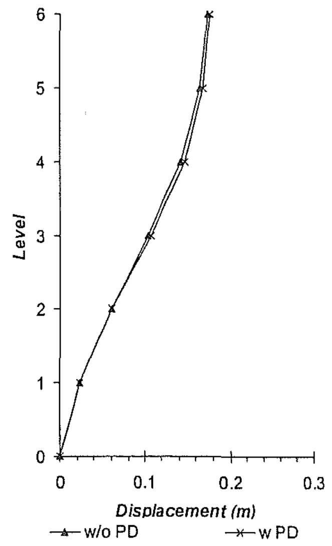
*El Centro 2*

*Maximum displacements*

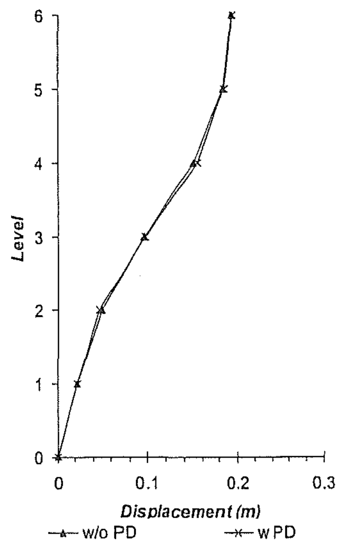
*6/6/E/T*




*6/6/3101/T*




*6/6/1170/T*

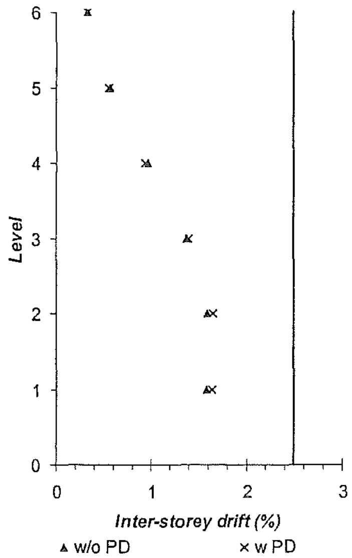


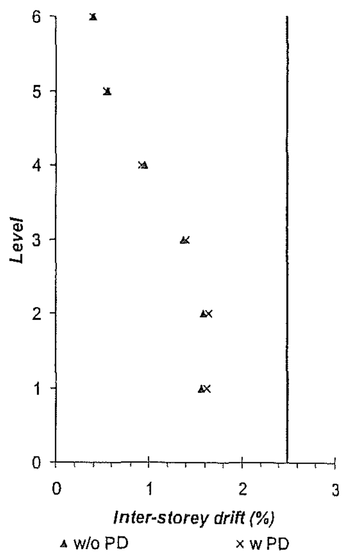

| | *6/6/E/T* | | *6/6/3101/T* | | *6/6/1170/T* | |
|---|---|---|---|---|---|---|
| | ***Maximum Displacements (m)*** | | ***Maximum Displacements (m)*** | | ***Maximum Displacements (m)*** | |
| ***Level*** | ***Without P-Δ*** | ***With P-Δ*** | ***Without P-Δ*** | ***With P-Δ*** | ***Without P-Δ*** | ***With P-Δ*** |
| ***1*** | 0.024 | 0.023 | 0.023 | 0.023 | 0.022 | 0.022 |
| ***2*** | 0.061 | 0.062 | 0.061 | 0.060 | 0.049 | 0.046 |
| ***3*** | 0.105 | 0.108 | 0.107 | 0.104 | 0.095 | 0.097 |
| ***4*** | 0.142 | 0.146 | 0.147 | 0.142 | 0.150 | 0.154 |
| ***5*** | 0.164 | 0.168 | 0.169 | 0.164 | 0.183 | 0.185 |
| ***6*** | 0.172 | 0.176 | 0.177 | 0.173 | 0.192 | 0.193 |
| ***Level*** | ***Central span beam rotation (rad)*** | | ***Central span beam rotation (rad)*** | | ***Central span beam rotation (rad)*** | |
| ***1*** | 0.0067 | | 0.0065 | | 0.0043 | |
| ***2*** | 0.0112 | | 0.0113 | | 0.0083 | |
| ***3*** | 0.0122 | | 0.0124 | | 0.0150 | |
| ***4*** | 0.0072 | | 0.0076 | | 0.0082 | |
| ***5*** | 0.0020 | | 0.0020 | | 0.0005 | |
| ***6*** | 0.0003 | | 0.0003 | | 0.0001 | |
| | ***Interior column rotation (rad)*** | | ***Interior column rotation (rad)*** | | ***Interior column rotation (rad)*** | |
| ***G. L.*** | 0.0051 | | 0.0052 | | 0.0048 | |
| ***1 Below*** | *Elastic columns at upper levels* | | 0.0009 | | 0.0008 | |
| ***1 Above*** | | | 0.0018 | | 0.0045 | |
| ***2 Below*** | | | 0.0013 | | 0.0026 | |
| ***2 Above*** | | | 0.0015 | | 0.0054 | |
| ***3 Below*** | | | 0.0011 | | 0.0022 | |
| ***3 Above*** | | | 0.0007 | | 0.0022 | |
| ***4 Below*** | | | 0.0014 | | 0.0089 | |
| ***4 Above*** | | | 0.0009 | | 0.0017 | |
| ***5 Below*** | | | 0.0013 | | 0.0084 | |
| ***5 Above*** | | | 0.0006 | | 0.0026 | |
| ***6 Below*** | | | 0.0014 | | 0.0040 | |

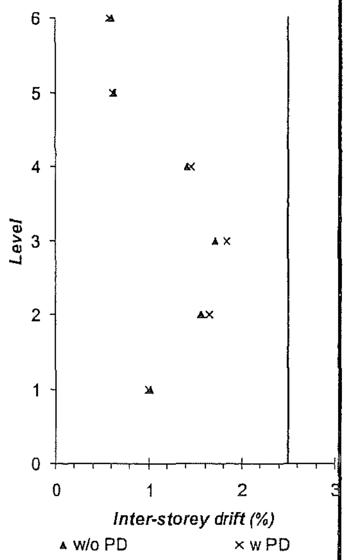
**Table B-7 Result comparison between models 6/6/E/T, 6/6/3101/T and 6/6/1170/T for Tabas 1 ground acceleration record**

| *Tabas 1* | | | | | | |
|---|---|---|---|---|---|---|
| | *6/6/E/T* | | *6/6/3101/T* | | *6/6/1170/T* | |
| | *Maximum base shear (kN)* | | *Maximum base shear (kN)* | | *Maximum base shear (kN)* | |
| *W/o P-Δ* | 1356.9 | | 1342.0 | | 1228.3 | |
| *With P-Δ* | 1542.5 | | 1530.1 | | 1327.4 | |

*Inter-storey drifts*

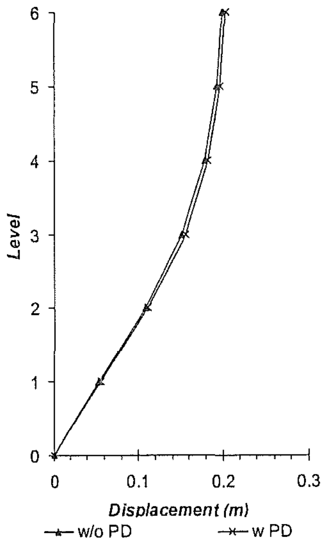
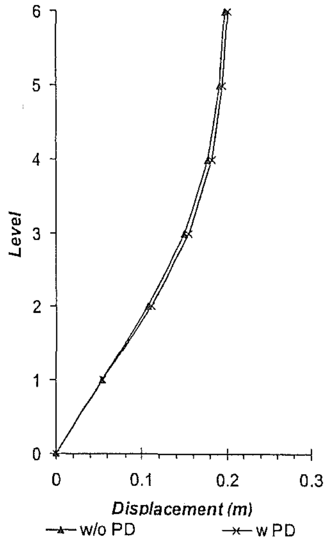
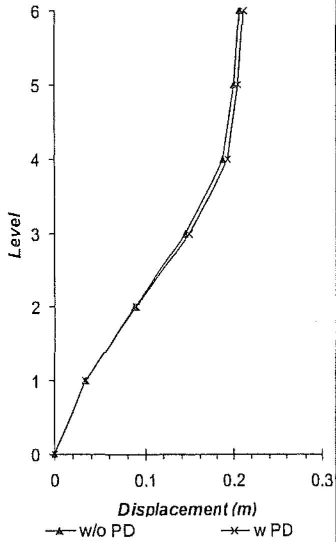



| | *Inter-storey drifts envelope (% of storey height)* | | *Inter-storey drifts envelope (% of storey height)* | | *Inter-storey drifts envelope (% of storey height)* | |
|---|---|---|---|---|---|---|
| *Storey* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* |
| *1* | 1.59 | 1.64 | 1.56 | 1.62 | 1.01 | 0.99 |
| *2* | 1.59 | 1.65 | 1.59 | 1.64 | 1.56 | 1.64 |
| *3* | 1.37 | 1.40 | 1.37 | 1.40 | 1.71 | 1.85 |
| *4* | 0.96 | 0.93 | 0.95 | 0.92 | 1.41 | 1.46 |
| *5* | 0.57 | 0.55 | 0.55 | 0.54 | 0.63 | 0.62 |
| *6* | 0.33 | 0.33 | 0.39 | 0.39 | 0.60 | 0.58 |

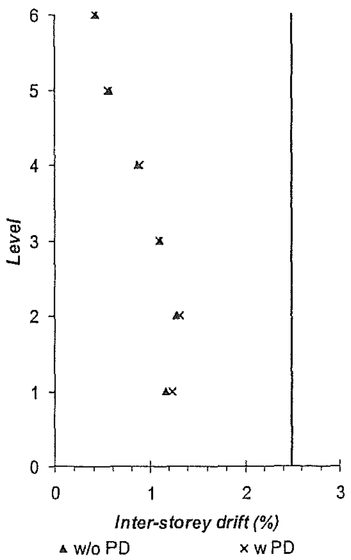
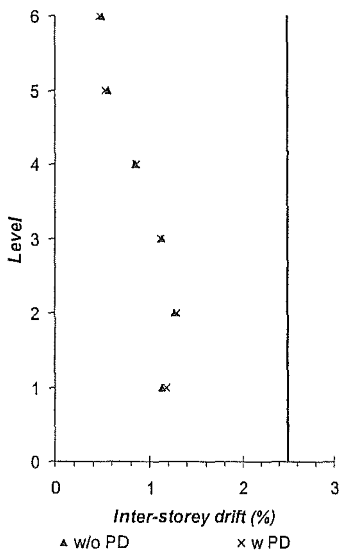
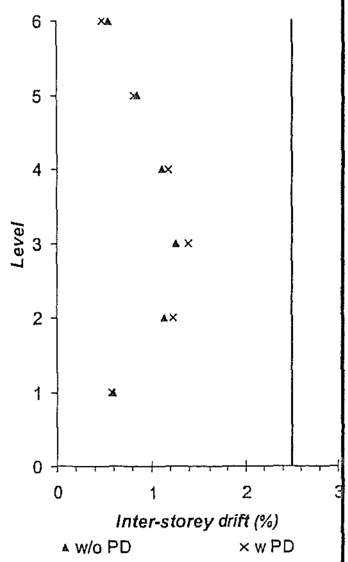
**Tabas 1**

| | 6/6/E/T | 6/6/3101/T | 6/6/1170/T |
|---|---|---|---|
| *Maximum displacements* |  |  |  |



| | 6/6/E/T | | 6/6/3101/T | | 6/6/1170/T | |
|---|---|---|---|---|---|---|
| | *Maximum Displacements (m)* | | *Maximum Displacements (m)* | | *Maximum Displacements (m)* | |
| *Level* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* |
| *1* | 0.054 | 0.056 | 0.053 | 0.055 | 0.034 | 0.034 |
| *2* | 0.108 | 0.112 | 0.107 | 0.111 | 0.087 | 0.089 |
| *3* | 0.151 | 0.155 | 0.150 | 0.155 | 0.143 | 0.148 |
| *4* | 0.178 | 0.183 | 0.178 | 0.182 | 0.187 | 0.193 |
| *5* | 0.192 | 0.196 | 0.192 | 0.196 | 0.200 | 0.205 |
| *6* | 0.199 | *0.203* | 0.199 | 0.202 | 0.207 | 0.211 |
| *Level* | *Central span beam rotation (rad)* | | *Central span beam rotation (rad)* | | *Central span beam rotation (rad)* | |
| *1* | 0.0151 | | 0.0148 | | 0.0083 | |
| *2* | 0.0135 | | 0.0135 | | 0.0156 | |
| *3* | 0.0106 | | 0.0105 | | 0.0149 | |
| *4* | 0.0044 | | 0.0045 | | 0.0028 | |
| *5* | 0.0027 | | 0.0027 | | 0.0005 | |
| *6* | 0.0004 | | 0.0003 | | 0.0001 | |
| | *Interior column rotation (rad)* | | *Interior column rotation (rad)* | | *Interior column rotation (rad)* | |
| *G. L.* | 0.0138 | | 0.0136 | | 0.0080 | |
| *1 Below* | | | 0.0009 | | 0.0009 | |
| *1 Above* | | | 0.0014 | | 0.0077 | |
| *2 Below* | | | 0.0014 | | 0.0013 | |
| *2 Above* | | | 0.0012 | | 0.0026 | |
| *3 Below* | | | 0.0013 | | 0.0038 | |
| *3 Above* | *Elastic columns at upper levels* | | 0.0010 | | 0.0025 | |
| *4 Below* | | | 0.0015 | | 0.0113 | |
| *4 Above* | | | 0.0006 | | 0.0045 | |
| *5 Below* | | | 0.0010 | | 0.0057 | |
| *5 Above* | | | 0.0005 | | 0.0055 | |
| *6 Below* | | | 0.0030 | | 0.0071 | |

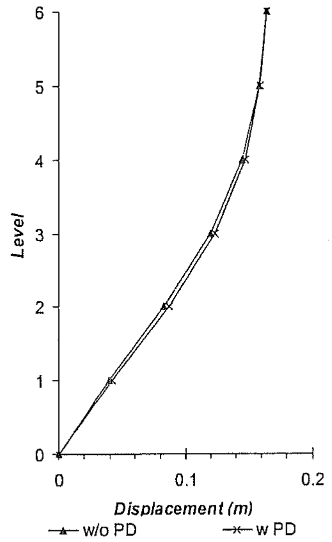
**Table B-8 Result comparison between models 6/6/E/T, 6/6/3101/T and 6/6/1170/T for Tabas 2 ground acceleration record**

| *Tabas 2* | | | | | | |
|---|---|---|---|---|---|---|
| | *6/6/E/T* | | *6/6/3101/T* | | *6/6/1170/T* | |
| | *Maximum base shear (kN)* | | *Maximum base shear (kN)* | | *Maximum base shear (kN)* | |
| *W/o P-Δ* | 1381.5 | | 1381.6 | | 1394.5 | |
| *With P-Δ* | 1355.8 | | 1356.7 | | 1368.7 | |
| *Inter-storey drifts* |   | |   | |   | |
| | *Inter-storey drifts envelope (% of storey height)* | | *Inter-storey drifts envelope (% of storey height)* | | *Inter-storey drifts envelope (% of storey height)* | |
| *Storey* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* |
| *1* | 1.16 | 1.24 | 1.13 | 1.17 | 0.58 | 0.57 |
| *2* | 1.28 | 1.31 | 1.26 | 1.29 | 1.13 | 1.22 |
| *3* | 1.10 | 1.09 | 1.13 | 1.11 | 1.26 | 1.39 |
| *4* | 0.86 | 0.88 | 0.84 | 0.86 | 1.10 | 1.17 |
| *5* | 0.57 | 0.55 | 0.57 | 0.53 | 0.85 | 0.81 |
| *6* | 0.42 | 0.41 | 0.49 | 0.47 | 0.54 | 0.47 |

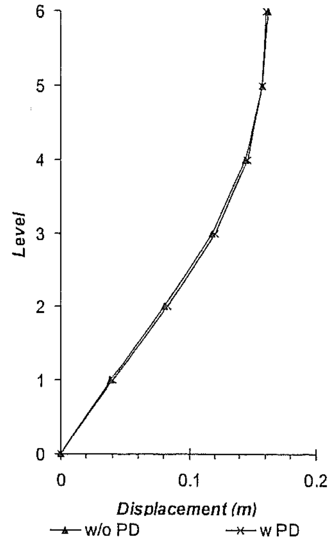
*Tabas 2*

*Maximum displacements*

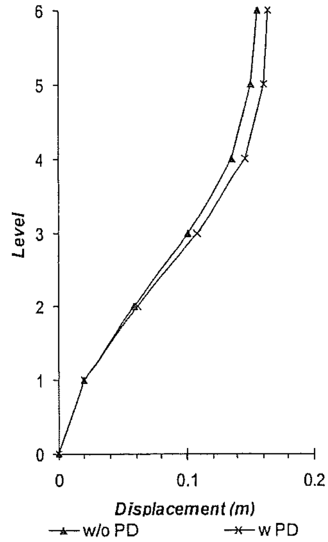
*6/6/E/T*




*6/6/3101/T*



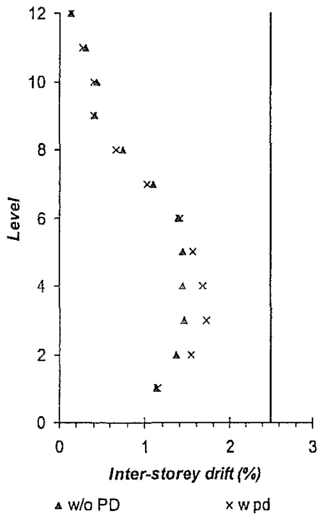
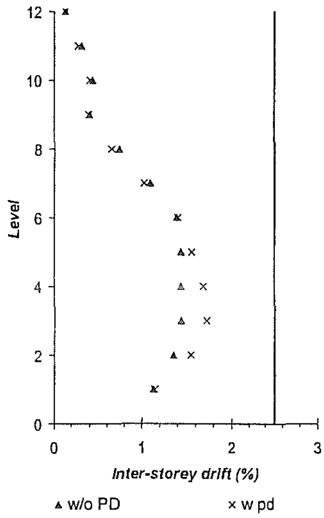
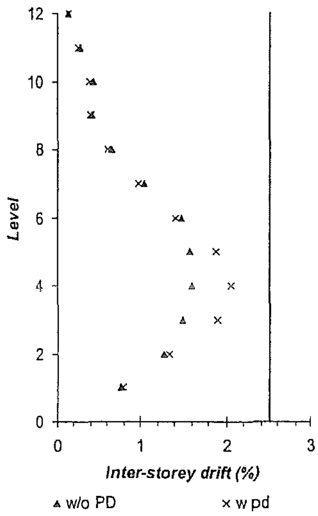

*6/6/1170/T*



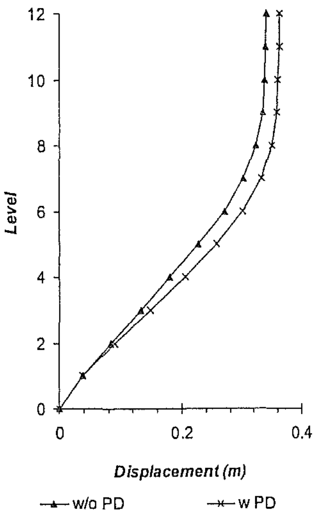
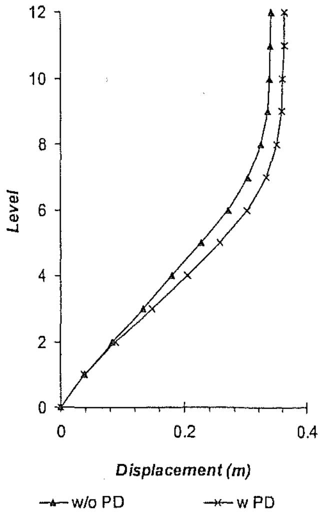
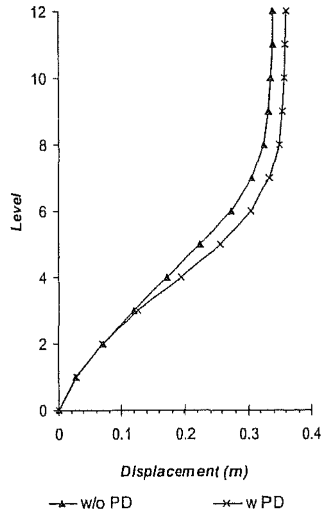

| | *6/6/E/T* | | *6/6/3101/T* | | *6/6/1170/T* | |
|---|---|---|---|---|---|---|
| | *Maximum Displacements (m)* | | *Maximum Displacements (m)* | | *Maximum Displacements (m)* | |
| *Level* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* |
| *1* | 0.039 | 0.042 | 0.038 | 0.040 | 0.020 | 0.0195 |
| *2* | 0.083 | 0.087 | 0.081 | 0.084 | 0.057 | 0.0605 |
| *3* | 0.120 | 0.123 | 0.119 | 0.121 | 0.100 | 0.1076 |
| *4* | 0.145 | 0.147 | 0.145 | 0.146 | 0.135 | 0.1457 |
| *5* | 0.159 | 0.159 | 0.159 | 0.158 | 0.150 | 0.1601 |
| *6* | 0.164 | 0.164 | 0.163 | 0.162 | 0.155 | 0.1637 |
| *Level* | *Central span beam rotation (rad)* | | *Central span beam rotation (rad)* | | *Central span beam rotation (rad)* | |
| *1* | 0.0107 | | 0.0104 | | 0.0040 | |
| *2* | 0.0102 | | 0.0101 | | 0.0108 | |
| *3* | 0.0077 | | 0.0080 | | 0.0103 | |
| *4* | 0.0060 | | 0.0057 | | 0.0028 | |
| *5* | 0.0034 | | 0.0032 | | 0.0005 | |
| *6* | 0.0005 | | 0.0002 | | 0.0001 | |
| | *Interior column rotation (rad)* | | *Interior column rotation (rad)* | | *Interior column rotation (rad)* | |
| *G. L.* | 0.0103 | | 0.0094 | | 0.0046 | |
| *1 Below* | | | 0.0010 | | 0.0010 | |
| *1 Above* | | | 0.0014 | | 0.0066 | |
| *2 Below* | | | 0.0013 | | 0.0013 | |
| *2 Above* | | | 0.0010 | | 0.0024 | |
| *3 Below* | | | 0.0012 | | 0.0045 | |
| *3 Above* | *Elastic columns at upper levels* | | 0.0013 | | 0.0033 | |
| *4 Below* | | | 0.0012 | | 0.0091 | |
| *4 Above* | | | 0.0006 | | 0.0056 | |
| *5 Below* | | | 0.0013 | | 0.0077 | |
| *5 Above* | | | 0.0007 | | 0.0035 | |
| *6 Below* | | | 0.0035 | | 0.0060 | |

## *B.2 Twelve Storey A Building*

**Table B-9 Results comparison between models 12A/6/E/T and 12A/6/3101/T and 12A/6/1170/T for Modified El Centro NS ground acceleration record**

| *Modified El Centro NS* | | | | | | |
|---|---|---|---|---|---|---|
| | *12A/6/E/T* | | *12A/6/3101/T* | | *12A/6/1170/T* | |
| | *Maximum base shear (kN)* | | *Maximum base shear (kN)* | | *Maximum base shear (kN)* | |
| *W/o P-Δ* | 1902.9 | | 1902.9 | | 1883.0 | |
| *With P-Δ* | 2056.1 | | 2056.1 | | 2016.8 | |
| *Inter-storey drifts* |  | |  | |  | |
| | *Maximum inter-storey drifts (% of storey height)* | | *Maximum inter-storey drifts (% of storey height)* | | *Maximum inter-storey drifts (% of storey height)* | |
| *Storey* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* |
| *1* | 1.126 | 1.144 | 1.126 | 1.136 | 0.76 | 0.80 |
| *2* | 1.358 | 1.532 | 1.358 | 1.536 | 1.28 | 1.34 |
| *3* | 1.440 | 1.717 | 1.440 | 1.718 | 1.49 | 1.90 |
| *4* | 1.432 | 1.679 | 1.432 | 1.682 | 1.61 | 2.05 |
| *5* | 1.435 | 1.550 | 1.435 | 1.553 | 1.57 | 1.88 |
| *6* | 1.388 | 1.394 | 1.388 | 1.394 | 1.48 | 1.41 |
| *7* | 1.091 | 1.015 | 1.091 | 1.015 | 1.04 | 0.97 |
| *8* | 0.735 | 0.656 | 0.735 | 0.656 | 0.65 | 0.61 |
| *9* | 0.409 | 0.397 | 0.409 | 0.397 | 0.41 | 0.40 |
| *10* | 0.438 | 0.403 | 0.438 | 0.403 | 0.43 | 0.39 |
| *11* | 0.303 | 0.271 | 0.303 | 0.271 | 0.28 | 0.25 |
| *12* | 0.135 | 0.135 | 0.135 | 0.135 | 0.13 | 0.13 |

*Modified El Centro NS*

| | *12A/6/E/T* | *12A/6/3101/T* | *12A/6/1170/T* |
|---|---|---|---|
| *Maximum displacements* |  |  |  |

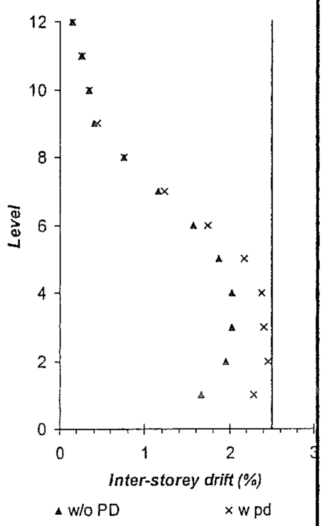
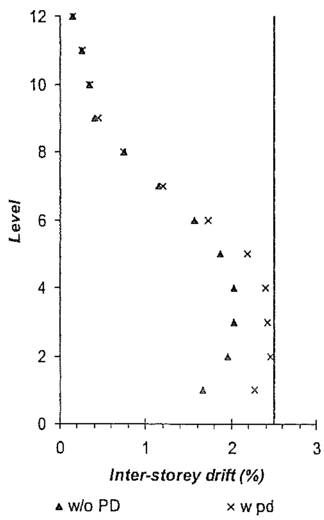
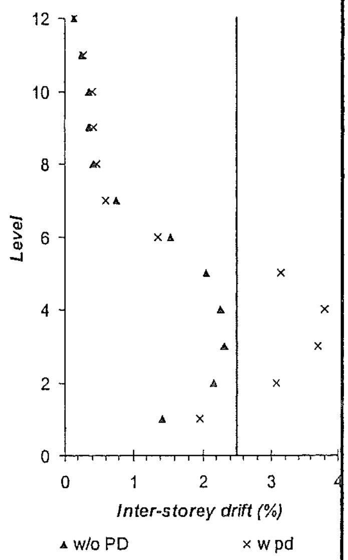


| *Level* | *Maximum displacements (m)* | | *Maximum displacements (m)* | | *Maximum displacements (m)* | |
|---|---|---|---|---|---|---|
| | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* |
| *1* | 0.038 | 0.039 | 0.038 | 0.039 | 0.026 | 0.027 |
| *2* | 0.084 | 0.090 | 0.084 | 0.090 | 0.069 | 0.071 |
| *3* | 0.132 | 0.149 | 0.132 | 0.148 | 0.119 | 0.125 |
| *4* | 0.179 | 0.206 | 0.179 | 0.206 | 0.171 | 0.195 |
| *5* | 0.227 | 0.258 | 0.227 | 0.258 | 0.224 | 0.257 |
| *6* | 0.270 | 0.301 | 0.270 | 0.301 | 0.272 | 0.304 |
| *7* | 0.303 | 0.332 | 0.303 | 0.332 | 0.306 | 0.333 |
| *8* | 0.323 | 0.350 | 0.323 | 0.350 | 0.325 | 0.347 |
| *9* | 0.334 | 0.357 | 0.334 | 0.358 | 0.332 | 0.353 |
| *10* | 0.338 | 0.360 | 0.338 | 0.361 | 0.335 | 0.356 |
| *11* | 0.339 | 0.362 | 0.339 | 0.362 | 0.337 | 0.358 |
| *12* | 0.340 | 0.364 | 0.340 | 0.364 | 0.339 | 0.359 |

| *Level* | *Central span beam rotations (rad)* | *Central span beam rotations (rad)* | *Central span beam rotations (rad)* |
|---|---|---|---|
| *1* | 0.0123 | 0.0123 | 0.0068 |
| *2* | 0.0158 | 0.0158 | 0.0134 |
| *3* | 0.0165 | 0.0166 | 0.0170 |
| *4* | 0.0156 | 0.0156 | 0.0106 |
| *5* | 0.0147 | 0.0147 | 0.0095 |
| *6* | 0.0115 | 0.0115 | 0.0087 |
| *7* | 0.0070 | 0.0070 | 0.0051 |
| *8* | 0.0034 | 0.0035 | 0.0020 |
| *9* | 0.0026 | 0.0027 | 0.0026 |
| *10* | 0.0017 | 0.0017 | 0.0014 |
| *11* | 0.0004 | 0.0005 | 0.0004 |
| *12* | 0.0002 | 0.0002 | 0.0002 |

| Modified El Centro NS | | | |
|---|---|---|---|
| | *12A/6/E/T* | *12A/6/3101/T* | *12A/6/1170/T* |
| | *Interior column rotation (rad)* | *Interior column rotation (rad)* | *Interior column rotation (rad)* |
| *G.L.* | 0.0083 | 0.0083 | 0.0057 |
| *1 Below* | *Elastic columns at upper levels* | 0.0008 | 0.0009 |
| *1 Above* | | 0.0018 | 0.0091 |
| *2 Below* | | 0.0013 | 0.0027 |
| *2 Above* | | 0.0014 | 0.0059 |
| *3 Below* | | 0.0012 | 0.0033 |
| *3 Above* | | 0.0014 | 0.0060 |
| *4 Below* | | 0.0012 | 0.0113 |
| *4 Above* | | 0.0011 | 0.0113 |
| *5 Below* | | 0.0011 | 0.0115 |
| *5 Above* | | 0.0007 | 0.0059 |
| *6 Below* | | 0.0013 | 0.0063 |
| *6 Above* | | 0.0008 | 0.0016 |
| *7 Below* | | 0.0013 | 0.0043 |
| *7 Above* | | 0.0008 | 0.0009 |
| *8 Below* | | 0.0012 | 0.0032 |
| *8 Above* | | 0.0009 | 0.0016 |
| *9 Below* | | 0.0010 | 0.0010 |
| *9 Above* | | 0.0007 | 0.0007 |
| *10 Below* | | 0.0009 | 0.0011 |
| *10 Above* | | 0.0005 | 0.0005 |
| *11 Below* | | 0.0008 | 0.0008 |
| *11 Above* | | 0.0003 | 0.0003 |
| *12 Below* | | 0.0004 | 0.0005 |

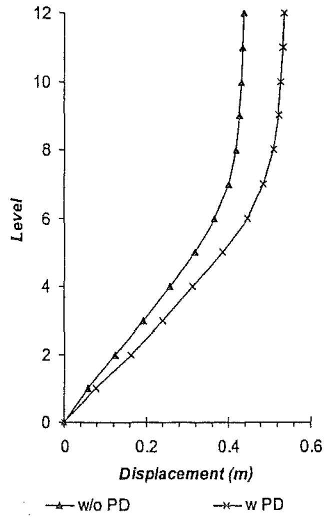
**Table B-10 Results comparison between models 12A/6/E/T, 12A/6/3101/T and 12A/6/1170/T for Modified El Centro EW ground acceleration record**

| *Modified El Centro EW* | | | | | | |
|---|---|---|---|---|---|---|
| | *12A/6/E/T* | | *12A/6/3101/T* | | *12A/6/1170/T* | |
| | *Maximum base shear (kN)* | | *Maximum base shear (kN)* | | *Maximum base shear (kN)* | |
| *W/o P-Δ* | 2001.7 | | 2001.4 | | 1934.7 | |
| *With P-Δ* | 2426.3 | | 2426.3 | | 2165.8 | |
| *Inter-storey drifts* |  | |  | |  | |
| | *Maximum inter-storey drifts (% of storey height)* | | *Maximum inter-storey drifts (% of storey height)* | | *Maximum inter-storey drifts (% of storey height)* | |
| *Storey* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* |
| *1* | 1.67 | 2.28 | 1.67 | 2.27 | 1.43 | 1.96 |
| *2* | 1.96 | 2.45 | 1.96 | 2.45 | 2.16 | 3.07 |
| *3* | 2.02 | 2.41 | 2.02 | 2.41 | 2.31 | 3.71 |
| *4* | 2.03 | 2.38 | 2.03 | 2.39 | 2.27 | 3.81 |
| *5* | 1.87 | 2.18 | 1.87 | 2.19 | 2.06 | 3.14 |
| *6* | 1.57 | 1.74 | 1.57 | 1.73 | 1.54 | 1.36 |
| *7* | 1.16 | 1.23 | 1.16 | 1.21 | 0.76 | 0.61 |
| *8* | 0.75 | 0.75 | 0.75 | 0.74 | 0.43 | 0.47 |
| *9* | 0.40 | 0.44 | 0.40 | 0.44 | 0.36 | 0.42 |
| *10* | 0.34 | 0.34 | 0.34 | 0.34 | 0.36 | 0.40 |
| *11* | 0.25 | 0.26 | 0.25 | 0.26 | 0.26 | 0.27 |
| *12* | 0.14 | 0.15 | 0.14 | 0.15 | 0.15 | 0.15 |

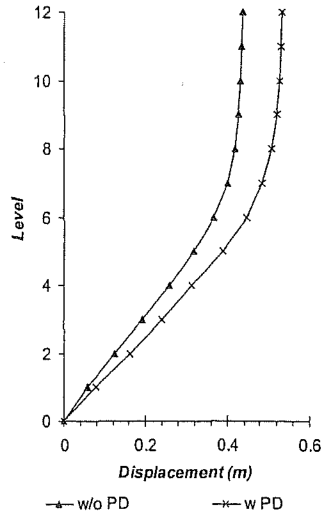
*Modified El Centro EW*

**Maximum displacements**

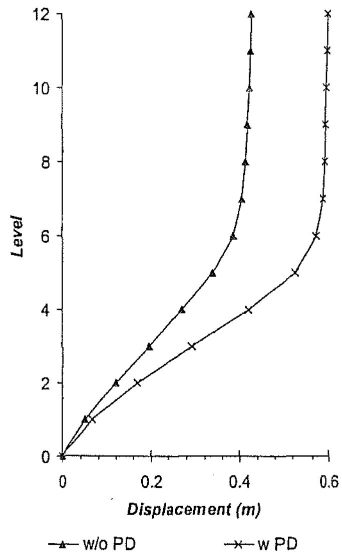
*12A/6/E/T*




*12A/6/3101/T*




*12A/6/1170/T*




| | 12A/6/E/T: Maximum displacements (m) | | 12A/6/3101/T: Maximum displacements (m) | | 12A/6/1170/T: Maximum displacements (m) | |
|---|---|---|---|---|---|---|
| *Level* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* |
| *1* | 0.057 | 0.077 | 0.057 | 0.077 | 0.049 | 0.067 |
| *2* | 0.123 | 0.161 | 0.123 | 0.161 | 0.121 | 0.169 |
| *3* | 0.191 | 0.239 | 0.191 | 0.239 | 0.197 | 0.291 |
| *4* | 0.256 | 0.313 | 0.256 | 0.313 | 0.270 | 0.420 |
| *5* | 0.318 | 0.387 | 0.318 | 0.387 | 0.337 | 0.525 |
| *6* | 0.365 | 0.445 | 0.366 | 0.446 | 0.384 | 0.568 |
| *7* | 0.401 | 0.485 | 0.401 | 0.484 | 0.404 | 0.584 |
| *8* | 0.419 | 0.509 | 0.419 | 0.508 | 0.411 | 0.590 |
| *9* | 0.427 | 0.521 | 0.427 | 0.520 | 0.416 | 0.594 |
| *10* | 0.431 | 0.527 | 0.431 | 0.526 | 0.420 | 0.596 |
| *11* | 0.433 | 0.531 | 0.433 | 0.530 | 0.423 | 0.598 |
| *12* | 0.435 | 0.534 | 0.435 | 0.533 | 0.425 | 0.600 |

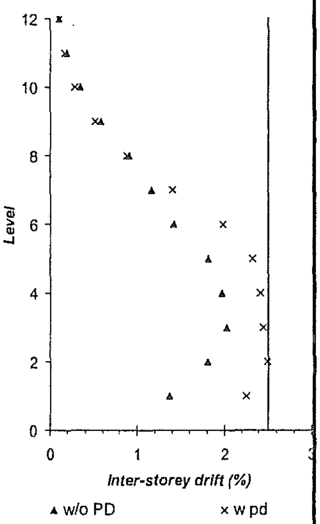
| *Level* | *Central span beam rotations (rad)* (12A/6/E/T) | *Central span beam rotations (rad)* (12A/6/3101/T) | *Central span beam rotations (rad)* (12A/6/1170/T) |
|---|---|---|---|
| *1* | 0.0238 | 0.0238 | 0.0184 |
| *2* | 0.0239 | 0.0240 | 0.0253 |
| *3* | 0.0243 | 0.0244 | 0.0298 |
| *4* | 0.0234 | 0.0235 | 0.0228 |
| *5* | 0.0200 | 0.0201 | 0.0136 |
| *6* | 0.0144 | 0.0141 | 0.0054 |
| *7* | 0.0088 | 0.0086 | 0.0036 |
| *8* | 0.0042 | 0.0041 | 0.0018 |
| *9* | 0.0022 | 0.0022 | 0.0026 |
| *10* | 0.0015 | 0.0015 | 0.0016 |
| *11* | 0.0005 | 0.0005 | 0.0005 |
| *12* | 0.0002 | 0.0002 | 0.0002 |

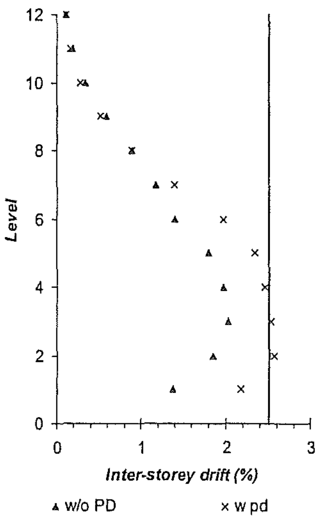
| | Modified El Centro EW | | |
|---|---|---|---|
| | *12A/6/E/T* | *12A/6/3101/T* | *12A/6/1170/T* |
| | ***Interior column rotation (rad)*** | ***Interior column rotation (rad)*** | ***Interior column rotation (rad)*** |
| *G.L.* | 0.0169 | 0.0168 | 0.0151 |
| ***1 Below*** | | 0.0008 | 0.0019 |
| ***1 Above*** | | 0.0016 | 0.0141 |
| ***2 Below*** | | 0.0013 | 0.0081 |
| ***2 Above*** | | 0.0012 | 0.0144 |
| ***3 Below*** | | 0.0014 | 0.0111 |
| ***3 Above*** | | 0.0010 | 0.0124 |
| ***4 Below*** | | 0.0015 | 0.0180 |
| ***4 Above*** | | 0.0008 | 0.0120 |
| ***5 Below*** | | 0.0016 | 0.0206 |
| ***5 Above*** | | 0.0008 | 0.0023 |
| ***6 Below*** | | 0.0016 | 0.0078 |
| ***6 Above*** | *Elastic columns at upper levels* | 0.0009 | 0.0020 |
| ***7 Below*** | | 0.0014 | 0.0034 |
| ***7 Above*** | | 0.0007 | 0.0009 |
| ***8 Below*** | | 0.0013 | 0.0019 |
| ***8 Above*** | | 0.0009 | 0.0018 |
| ***9 Below*** | | 0.0010 | 0.0008 |
| ***9 Above*** | | 0.0007 | 0.0008 |
| ***10 Below*** | | 0.0009 | 0.0012 |
| ***10 Above*** | | 0.0006 | 0.0006 |
| ***11 Below*** | | 0.0008 | 0.0008 |
| ***11 Above*** | | 0.0003 | 0.0004 |
| ***12 Below*** | | 0.0005 | 0.0006 |

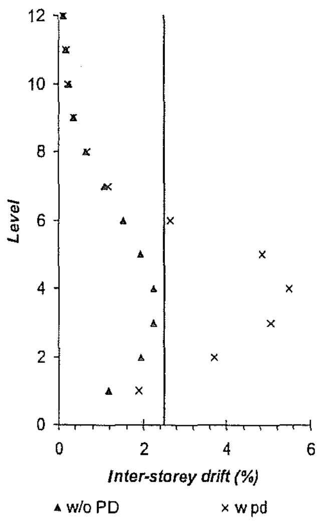
**Table B-11 Results comparison between models 12A/6/E/T, 12A/6/3101/T and 12A/6/1170/T for Modified TAFT 1 ground acceleration record**

| *Modified TAFT 1* | | | | | | |
|---|---|---|---|---|---|---|
| | *12A/6/E/T* | | *12A/6/3101/T* | | *12A/6/1170/T* | |
| | *Maximum base shear (kN)* | | *Maximum base shear (kN)* | | *Maximum base shear (kN)* | |
| *W/o P-Δ* | 1812.3 | | 1805.2 | | 1859.5 | |
| *With P-Δ* | 2431.8 | | 2395.7 | | 2142.9 | |

*Inter-storey drifts*

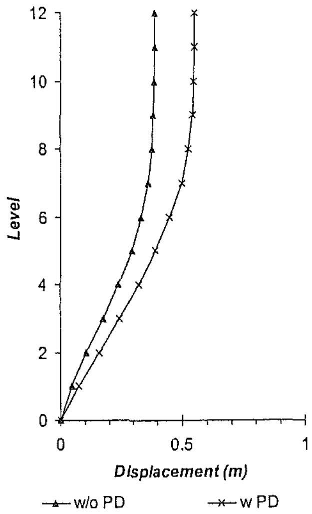
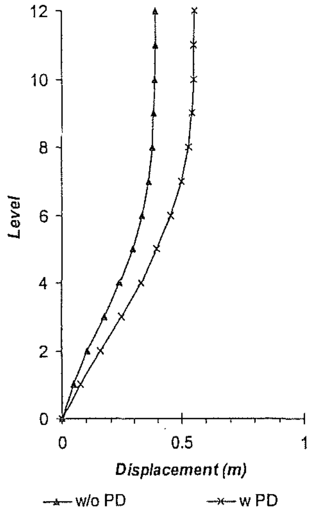
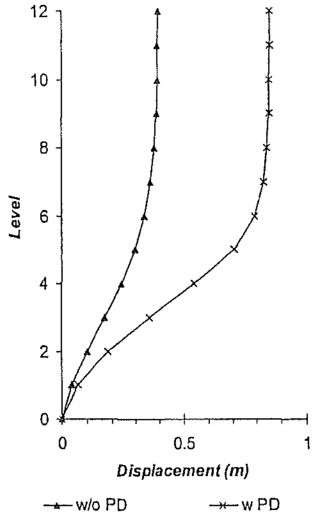


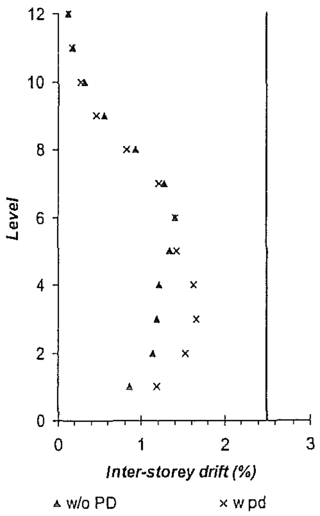
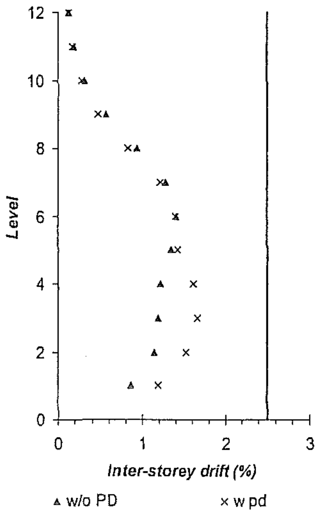
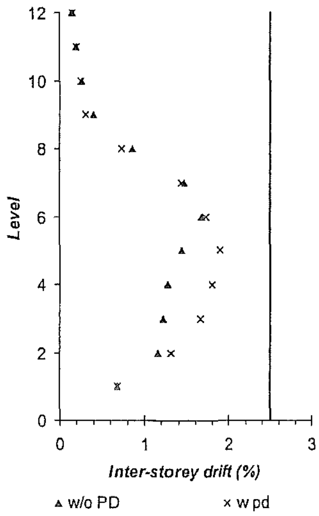

| | *12A/6/E/T* | | *12A/6/3101/T* | | *12A/6/1170/T* | |
|---|---|---|---|---|---|---|
| | *Maximum inter-storey drifts (% of storey height)* | | *Maximum inter-storey drifts (% of storey height)* | | *Maximum inter-storey drifts (% of storey height)* | |
| *Storey* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* |
| *1* | 1.37 | 2.26 | 1.368 | 2.18 | 1.16 | 1.87 |
| *2* | 1.81 | 2.49 | 1.84 | 2.56 | 1.92 | 3.69 |
| *3* | 2.03 | 2.44 | 2.03 | 2.52 | 2.24 | 5.06 |
| *4* | 1.97 | 2.41 | 1.97 | 2.45 | 2.24 | 5.48 |
| *5* | 1.81 | 2.32 | 1.80 | 2.34 | 1.92 | 4.86 |
| *6* | 1.41 | 1.98 | 1.40 | 1.97 | 1.52 | 2.65 |
| *7* | 1.16 | 1.40 | 1.17 | 1.39 | 1.07 | 1.15 |
| *8* | 0.90 | 0.88 | 0.89 | 0.89 | 0.64 | 0.65 |
| *9* | 0.58 | 0.51 | 0.59 | 0.51 | 0.34 | 0.33 |
| *10* | 0.35 | 0.28 | 0.34 | 0.28 | 0.24 | 0.23 |
| *11* | 0.19 | 0.16 | 0.19 | 0.16 | 0.18 | 0.18 |
| *12* | 0.11 | 0.11 | 0.11 | 0.11 | 0.12 | 0.12 |

*Modified TAFT 1*

| | *12A/6/E/T* | | *12A/6/3101/T* | | *12A/6/1170/T* | |
|---|---|---|---|---|---|---|
| *Maximum displacements* |  | |  | |  | |
| | *Maximum displacements (m)* | | *Maximum displacements (m)* | | *Maximum displacements (m)* | |
| *Level* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* |
| *1* | 0.047 | 0.077 | 0.047 | 0.074 | 0.040 | 0.064 |
| *2* | 0.107 | 0.162 | 0.107 | 0.161 | 0.103 | 0.187 |
| *3* | 0.176 | 0.245 | 0.176 | 0.247 | 0.173 | 0.356 |
| *4* | 0.241 | 0.322 | 0.241 | 0.326 | 0.243 | 0.542 |
| *5* | 0.294 | 0.390 | 0.294 | 0.396 | 0.298 | 0.705 |
| *6* | 0.333 | 0.451 | 0.333 | 0.454 | 0.337 | 0.792 |
| *7* | 0.360 | 0.498 | 0.360 | 0.501 | 0.363 | 0.826 |
| *8* | 0.376 | 0.527 | 0.376 | 0.529 | 0.378 | 0.842 |
| *9* | 0.384 | 0.541 | 0.384 | 0.543 | 0.385 | 0.847 |
| *10* | 0.388 | 0.547 | 0.388 | 0.550 | 0.389 | 0.849 |
| *11* | 0.389 | 0.550 | 0.389 | 0.553 | 0.391 | 0.852 |
| *12* | 0.390 | 0.552 | 0.390 | 0.555 | 0.393 | 0.853 |
| ***Level*** | ***Central span beam rotations (rad)*** | | ***Central span beam rotations (rad)*** | | ***Central span beam rotations (rad)*** | |
| *1* | 0.0239 | | 0.0227 | | 0.0176 | |
| *2* | 0.0250 | | 0.0261 | | 0.0264 | |
| *3* | 0.0241 | | 0.0248 | | 0.0322 | |
| *4* | 0.0242 | | 0.0246 | | 0.0245 | |
| *5* | 0.0222 | | 0.0222 | | 0.0196 | |
| *6* | 0.0169 | | 0.0168 | | 0.0098 | |
| *7* | 0.0099 | | 0.0098 | | 0.0057 | |
| *8* | 0.0054 | | 0.0055 | | 0.0018 | |
| *9* | 0.0020 | | 0.0020 | | 0.0008 | |
| *10* | 0.0005 | | 0.0005 | | 0.0005 | |
| *11* | 0.0003 | | 0.0003 | | 0.0004 | |
| *12* | 0.0002 | | 0.0002 | | 0.0002 | |

| *Modified TAFT 1* | | | |
|---|---|---|---|
| | *12A/6/E/T* | *12A/6/3101/T* | *12A/6/1170/T* |
| | ***Interior column rotation (rad)*** | ***Interior column rotation (rad)*** | ***Interior column rotation (rad)*** |
| *G.L.* | 0.0162 | 0.0158 | 0.0140 |
| ***1 Below*** | *Elastic columns at upper levels* | 0.0008 | 0.0016 |
| ***1 Above*** | | 0.0036 | 0.0224 |
| ***2 Below*** | | 0.0013 | 0.0140 |
| ***2 Above*** | | 0.0012 | 0.0281 |
| ***3 Below*** | | 0.0013 | 0.0223 |
| ***3 Above*** | | 0.0011 | 0.0278 |
| ***4 Below*** | | 0.0015 | 0.0344 |
| ***4 Above*** | | 0.0010 | 0.0269 |
| ***5 Below*** | | 0.0016 | 0.0365 |
| ***5 Above*** | | 0.0008 | 0.0102 |
| ***6 Below*** | | 0.0016 | 0.0191 |
| ***6 Above*** | | 0.0008 | 0.0022 |
| ***7 Below*** | | 0.0016 | 0.0058 |
| ***7 Above*** | | 0.0007 | 0.0009 |
| ***8 Below*** | | 0.0013 | 0.0038 |
| ***8 Above*** | | 0.0007 | 0.0007 |
| ***9 Below*** | | 0.0011 | 0.0012 |
| ***9 Above*** | | 0.0006 | 0.0006 |
| ***10 Below*** | | 0.0008 | 0.0007 |
| ***10 Above*** | | 0.0004 | 0.0005 |
| ***11 Below*** | | 0.0005 | 0.0005 |
| ***11 Above*** | | 0.0003 | 0.0003 |
| ***12 Below*** | | 0.0004 | 0.0004 |

**Table B-12 Result comparison between models 12A/6/E/T, 12A/6/3101/T and 12A/6/1170/T for Modified TAFT 2 ground acceleration record**

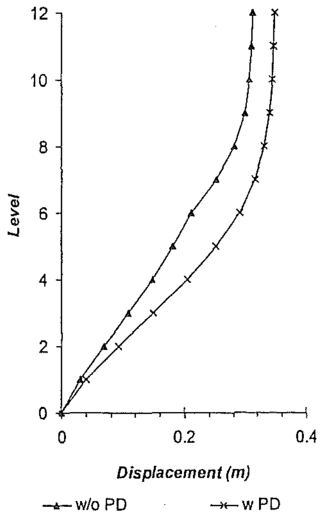
| *Modified TAFT 2* | | | | | | |
|---|---|---|---|---|---|---|
| | *12A/6/E/T* | | *12A/6/3101/T* | | *12A/6/1170/T* | |
| | *Maximum base shear (kN)* | | *Maximum base shear (kN)* | | *Maximum base shear (kN)* | |
| *W/o P-Δ* | 1781.4 | | 1776.2 | | 1781.0 | |
| *With P-Δ* | 1951.3 | | 1951.1 | | 1892.3 | |
| *Inter-storey drifts* |  | |  | |  | |
| | *Maximum inter-storey drifts (% of storey height)* | | *Maximum inter-storey drifts (% of storey height)* | | *Maximum inter-storey drifts (% of storey height)* | |
| *Storey* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* |
| *1* | 0.87 | 1.20 | 0.866 | 1.19 | 0.68 | 0.68 |
| *2* | 1.15 | 1.53 | 1.14 | 1.53 | 1.16 | 1.31 |
| *3* | 1.19 | 1.67 | 1.19 | 1.66 | 1.23 | 1.66 |
| *4* | 1.22 | 1.62 | 1.22 | 1.62 | 1.28 | 1.80 |
| *5* | 1.34 | 1.42 | 1.34 | 1.42 | 1.43 | 1.90 |
| *6* | 1.41 | 1.40 | 1.41 | 1.40 | 1.68 | 1.72 |
| *7* | 1.28 | 1.21 | 1.28 | 1.21 | 1.47 | 1.43 |
| *8* | 0.94 | 0.83 | 0.93 | 0.83 | 0.86 | 0.72 |
| *9* | 0.56 | 0.46 | 0.55 | 0.46 | 0.40 | 0.31 |
| *10* | 0.32 | 0.28 | 0.32 | 0.28 | 0.25 | 0.24 |
| *11* | 0.18 | 0.17 | 0.18 | 0.17 | 0.20 | 0.19 |
| *12* | 0.12 | 0.12 | 0.12 | 0.12 | 0.15 | 0.15 |

## Modified TAFT 2

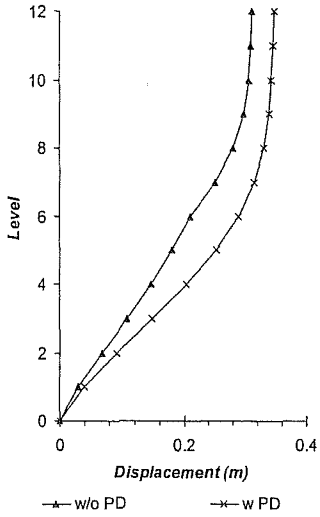
**Maximum displacements**

| | 12A/6/E/T | 12A/6/3101/T | 12A/6/1170/T |
|---|---|---|---|

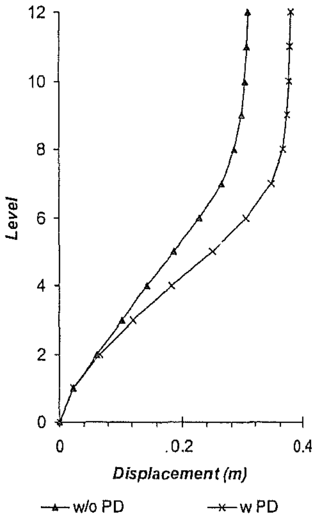
12A/6/E/T




12A/6/3101/T




12A/6/1170/T

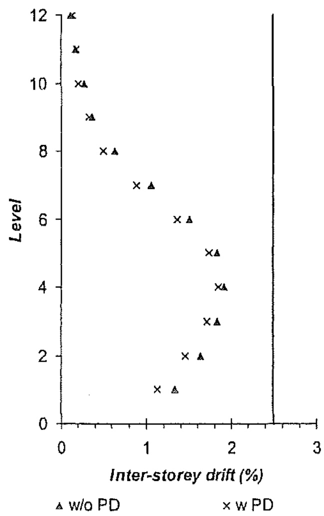
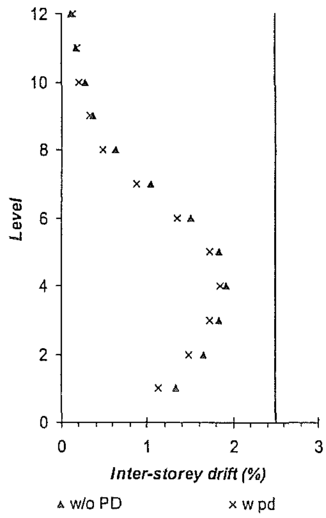
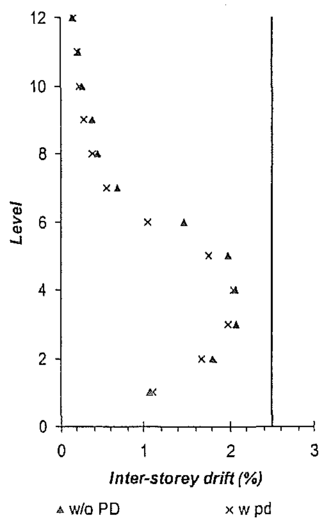



| | 12A/6/E/T Maximum displacements (m) | | 12A/6/3101/T Maximum displacements (m) | | 12A/6/1170/T Maximum displacements (m) | |
|---|---|---|---|---|---|---|
| *Level* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* |
| *1* | 0.029 | 0.041 | 0.029 | 0.041 | 0.023 | 0.023 |
| *2* | 0.068 | 0.093 | 0.068 | 0.093 | 0.062 | 0.066 |
| *3* | 0.109 | 0.149 | 0.109 | 0.149 | 0.104 | 0.122 |
| *4* | 0.147 | 0.205 | 0.147 | 0.204 | 0.144 | 0.183 |
| *5* | 0.182 | 0.253 | 0.181 | 0.252 | 0.187 | 0.248 |
| *6* | 0.211 | 0.290 | 0.211 | 0.290 | 0.229 | 0.305 |
| *7* | 0.252 | 0.316 | 0.251 | 0.315 | 0.264 | 0.347 |
| *8* | 0.282 | 0.331 | 0.281 | 0.331 | 0.286 | 0.365 |
| *9* | 0.300 | 0.340 | 0.299 | 0.339 | 0.298 | 0.371 |
| *10* | 0.308 | 0.345 | 0.307 | 0.344 | 0.303 | 0.375 |
| *11* | 0.311 | 0.347 | 0.310 | 0.346 | 0.306 | 0.377 |
| *12* | 0.313 | 0.348 | 0.312 | 0.348 | 0.308 | 0.378 |

| *Level* | *Central span beam rotations (rad)* 12A/6/E/T | *Central span beam rotations (rad)* 12A/6/3101/T | *Central span beam rotations (rad)* 12A/6/1170/T |
|---|---|---|---|
| *1* | 0.0125 | 0.0125 | 0.0054 |
| *2* | 0.0153 | 0.0154 | 0.0127 |
| *3* | 0.0159 | 0.0159 | 0.0159 |
| *4* | 0.0148 | 0.0146 | 0.0140 |
| *5* | 0.0137 | 0.0137 | 0.0139 |
| *6* | 0.0128 | 0.0129 | 0.0136 |
| *7* | 0.0095 | 0.0095 | 0.0070 |
| *8* | 0.0046 | 0.0046 | 0.0020 |
| *9* | 0.0018 | 0.0018 | 0.0008 |
| *10* | 0.0005 | 0.0005 | 0.0005 |
| *11* | 0.0004 | 0.0004 | 0.0004 |
| *12* | 0.0002 | 0.0002 | 0.0002 |

| Modified TAFT 2 | | | |
|---|---|---|---|
| | 12A/6/E/T | 12A/6/3101/T | 12A/6/1170/T |
| | Interior column rotation (rad) | Interior column rotation (rad) | Interior column rotation (rad) |
| G.L. | 0.0080 | 0.0081 | 0.0050 |
| 1 Below | Elastic columns at upper levels | 0.0008 | 0.0009 |
| 1 Above | | 0.0016 | 0.0076 |
| 2 Below | | 0.0011 | 0.0023 |
| 2 Above | | 0.0014 | 0.0040 |
| 3 Below | | 0.0011 | 0.0030 |
| 3 Above | | 0.0013 | 0.0041 |
| 4 Below | | 0.0012 | 0.0069 |
| 4 Above | | 0.0012 | 0.0071 |
| 5 Below | | 0.0012 | 0.0076 |
| 5 Above | | 0.0010 | 0.0045 |
| 6 Below | | 0.0013 | 0.0055 |
| 6 Above | | 0.0007 | 0.0020 |
| 7 Below | | 0.0013 | 0.0076 |
| 7 Above | | 0.0007 | 0.0008 |
| 8 Below | | 0.0013 | 0.0041 |
| 8 Above | | 0.0007 | 0.0008 |
| 9 Below | | 0.0011 | 0.0009 |
| 9 Above | | 0.0007 | 0.0007 |
| 10 Below | | 0.0008 | 0.0008 |
| 10 Above | | 0.0005 | 0.0005 |
| 11 Below | | 0.0006 | 0.0006 |
| 11 Above | | 0.0003 | 0.0004 |
| 12 Below | | 0.0004 | 0.0005 |

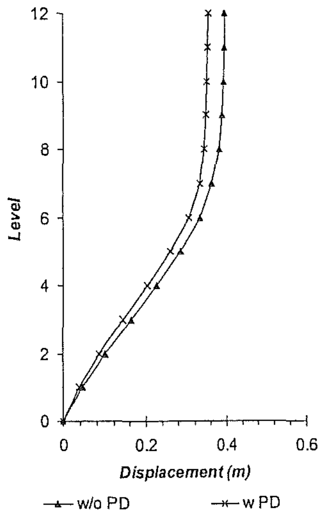
**Table B-13 Result comparison between models 12A/6/E/T, 12A/6/3101/T and 12A/6/1170/T for El Centro 1 ground acceleration record**

| *El Centro 1* | | | | | | |
|---|---|---|---|---|---|---|
| | *12A/6/E/T* | | *12A/6/3101/T* | | *12A/6/1170/T* | |
| | *Maximum base shear (kN)* | | *Maximum base shear (kN)* | | *Maximum base shear (kN)* | |
| *W/o P-Δ* | 1872.0 | | 1872.0 | | 1875.6 | |
| *With P-Δ* | 2047.6 | | 2047.6 | | 2043.7 | |
| *Inter-storey drifts* |  | |  | |  | |
| | *Maximum inter-storey drifts (% of storey height)* | | *Maximum inter-storey drifts (% of storey height)* | | *Maximum inter-storey drifts (% of storey height)* | |
| *Storey* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* |
| *1* | 1.33 | 1.13 | 1.33 | 1.13 | 1.07 | 1.11 |
| *2* | 1.64 | 1.45 | 1.65 | 1.48 | 1.81 | 1.67 |
| *3* | 1.83 | 1.72 | 1.83 | 1.73 | 2.08 | 1.98 |
| *4* | 1.92 | 1.85 | 1.92 | 1.84 | 2.06 | 2.04 |
| *5* | 1.84 | 1.74 | 1.84 | 1.72 | 1.98 | 1.76 |
| *6* | 1.51 | 1.36 | 1.51 | 1.35 | 1.47 | 1.04 |
| *7* | 1.06 | 0.89 | 1.05 | 0.87 | 0.68 | 0.55 |
| *8* | 0.63 | 0.50 | 0.63 | 0.49 | 0.44 | 0.38 |
| *9* | 0.37 | 0.33 | 0.36 | 0.33 | 0.37 | 0.28 |
| *10* | 0.28 | 0.21 | 0.28 | 0.21 | 0.26 | 0.23 |
| *11* | 0.17 | 0.19 | 0.17 | 0.19 | 0.22 | 0.21 |
| *12* | 0.11 | 0.13 | 0.11 | 0.13 | 0.15 | 0.16 |

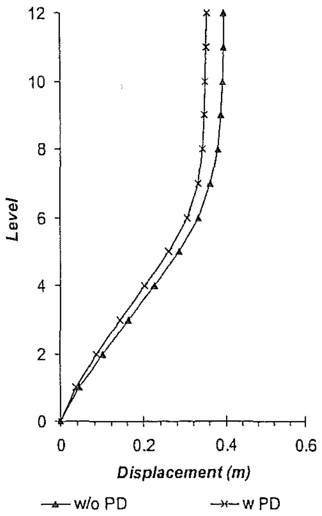
## El Centro 1

Maximum displacements

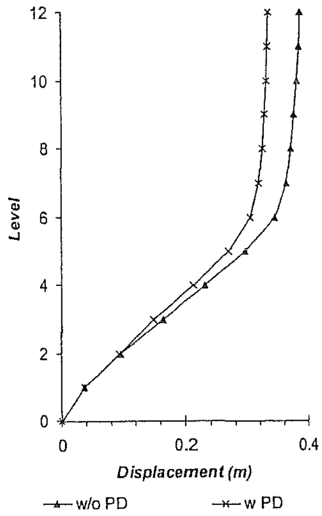
12A/6/E/T




12A/6/3101/T




12A/6/1170/T

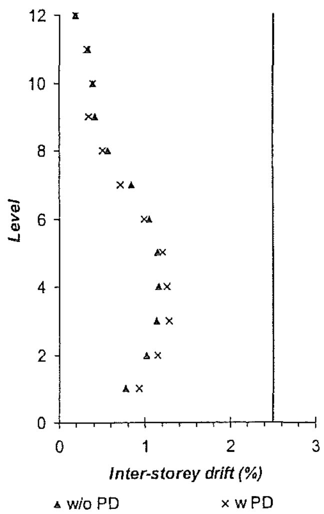
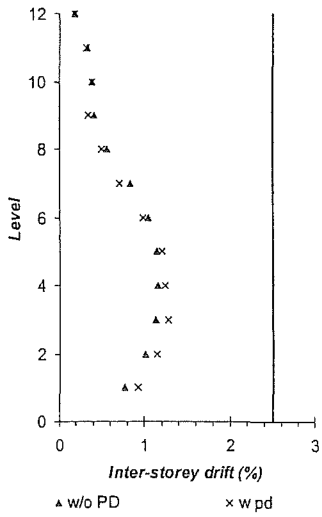
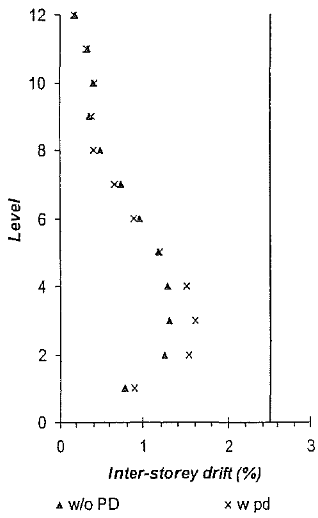



| | 12A/6/E/T | | 12A/6/3101/T | | 12A/6/1170/T | |
|---|---|---|---|---|---|---|
| | *Maximum displacements (m)* | | *Maximum displacements (m)* | | *Maximum displacements (m)* | |
| *Level* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* |
| *1* | 0.045 | 0.038 | 0.045 | 0.038 | 0.036 | 0.038 |
| *2* | 0.101 | 0.086 | 0.101 | 0.086 | 0.096 | 0.094 |
| *3* | 0.162 | 0.141 | 0.162 | 0.141 | 0.164 | 0.147 |
| *4* | 0.224 | 0.202 | 0.224 | 0.202 | 0.230 | 0.212 |
| *5* | 0.284 | 0.259 | 0.284 | 0.260 | 0.295 | 0.269 |
| *6* | 0.333 | 0.305 | 0.333 | 0.305 | 0.343 | 0.305 |
| *7* | 0.364 | 0.332 | 0.364 | 0.332 | 0.363 | 0.317 |
| *8* | 0.382 | 0.343 | 0.382 | 0.343 | 0.370 | 0.323 |
| *9* | 0.389 | 0.348 | 0.388 | 0.348 | 0.375 | 0.327 |
| *10* | 0.392 | 0.352 | 0.392 | 0.351 | 0.380 | 0.329 |
| *11* | 0.395 | 0.354 | 0.395 | 0.354 | 0.383 | 0.331 |
| *12* | 0.397 | 0.355 | 0.397 | 0.355 | 0.385 | 0.333 |

| *Level* | *Central span beam rotations (rad)* | *Central span beam rotations (rad)* | *Central span beam rotations (rad)* |
|---|---|---|---|
| *1* | 0.0116 | 0.0116 | 0.0100 |
| *2* | 0.0153 | 0.0155 | 0.0162 |
| *3* | 0.0176 | 0.0176 | 0.0175 |
| *4* | 0.0182 | 0.0180 | 0.0156 |
| *5* | 0.0155 | 0.0153 | 0.0106 |
| *6* | 0.0106 | 0.0104 | 0.0044 |
| *7* | 0.0050 | 0.0048 | 0.0024 |
| *8* | 0.0024 | 0.0023 | 0.0014 |
| *9* | 0.0007 | 0.0007 | 0.0006 |
| *10* | 0.0005 | 0.0005 | 0.0005 |
| *11* | 0.0004 | 0.0004 | 0.0004 |
| *12* | 0.0002 | 0.0002 | 0.0002 |

| El Centro 1 | | | |
|---|---|---|---|
| | 12A/6/E/T | 12A/6/3101/T | 12A/6/1170/T |
| | Interior column rotation (rad) | Interior column rotation (rad) | Interior column rotation (rad) |
| G.L. | 0.0077 | 0.0077 | 0.0080 |
| 1 Below | Elastic columns at upper levels | 0.0009 | 0.0010 |
| 1 Above | | 0.0022 | 0.0086 |
| 2 Below | | 0.0013 | 0.0061 |
| 2 Above | | 0.0013 | 0.0053 |
| 3 Below | | 0.0012 | 0.0052 |
| 3 Above | | 0.0011 | 0.0039 |
| 4 Below | | 0.0013 | 0.0078 |
| 4 Above | | 0.0008 | 0.0046 |
| 5 Below | | 0.0014 | 0.0078 |
| 5 Above | | 0.0008 | 0.0033 |
| 6 Below | | 0.0015 | 0.0069 |
| 6 Above | | 0.0007 | 0.0009 |
| 7 Below | | 0.0014 | 0.0023 |
| 7 Above | | 0.0007 | 0.0008 |
| 8 Below | | 0.0011 | 0.0012 |
| 8 Above | | 0.0006 | 0.0007 |
| 9 Below | | 0.0009 | 0.0008 |
| 9 Above | | 0.0006 | 0.0007 |
| 10 Below | | 0.0007 | 0.0007 |
| 10 Above | | 0.0006 | 0.0006 |
| 11 Below | | 0.0006 | 0.0007 |
| 11 Above | | 0.0004 | 0.0004 |
| 12 Below | | 0.0004 | 0.0006 |

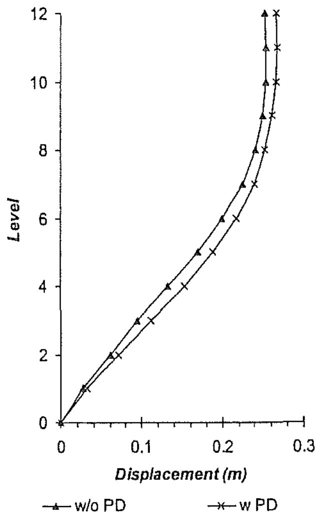
**Table B-14 Result comparison between models 12A/6/E/T, 12A/6/3101/T and 12A/6/1170/T for El Centro 2 ground acceleration record**

| *El Centro 2* | | | | | | |
|---|---|---|---|---|---|---|
| | *12A/6/E/T* | | *12A/6/3101/T* | | *12A/6/1170/T* | |
| | *Maximum base shear (kN)* | | *Maximum base shear (kN)* | | *Maximum base shear (kN)* | |
| *W/o P-Δ* | 1838.1 | | 1838.1 | | 1813.4 | |
| *With P-Δ* | 2061.0 | | 2061.0 | | 2030.8 | |
| *Inter-storey drifts* |   | |   | |   | |
| | *Maximum inter-storey drifts (% of storey height)* | | *Maximum inter-storey drifts (% of storey height)* | | *Maximum inter-storey drifts (% of storey height)* | |
| *Storey* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* |
| *1* | 0.77 | 0.93 | 0.771 | 0.93 | 0.76 | 0.89 |
| *2* | 1.01 | 1.15 | 1.01 | 1.15 | 1.24 | 1.51 |
| *3* | 1.13 | 1.28 | 1.13 | 1.28 | 1.30 | 1.60 |
| *4* | 1.15 | 1.25 | 1.15 | 1.24 | 1.28 | 1.49 |
| *5* | 1.15 | 1.20 | 1.15 | 1.20 | 1.16 | 1.18 |
| *6* | 1.05 | 0.99 | 1.05 | 0.99 | 0.96 | 0.88 |
| *7* | 0.84 | 0.70 | 0.84 | 0.70 | 0.73 | 0.66 |
| *8* | 0.56 | 0.50 | 0.56 | 0.50 | 0.49 | 0.41 |
| *9* | 0.41 | 0.34 | 0.41 | 0.34 | 0.35 | 0.37 |
| *10* | 0.39 | 0.39 | 0.39 | 0.39 | 0.41 | 0.42 |
| *11* | 0.33 | 0.32 | 0.33 | 0.32 | 0.32 | 0.31 |
| *12* | 0.19 | 0.18 | 0.19 | 0.18 | 0.18 | 0.17 |

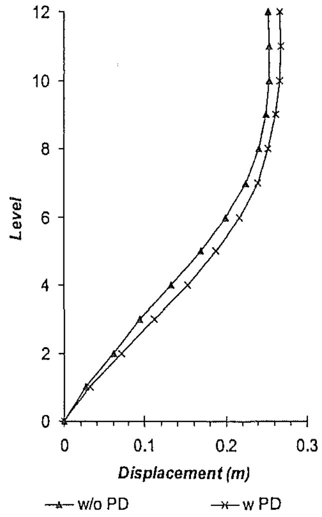
## *El Centro 2*

*Maximum displacements*

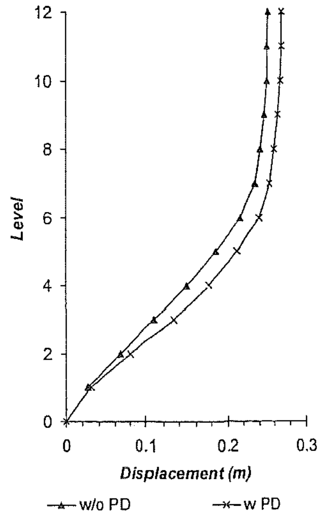
*12A/6/E/T*




*12A/6/3101/T*




*12A/6/1170/T*

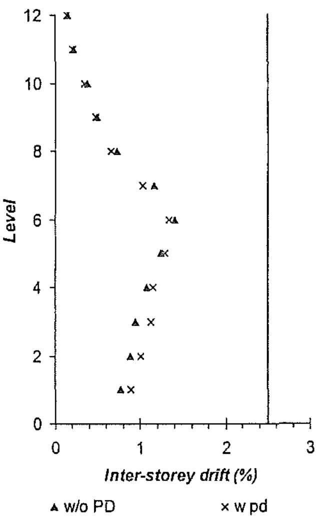
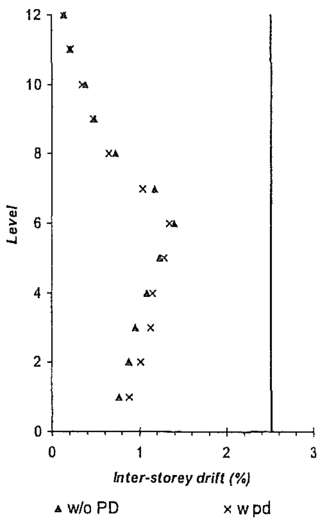
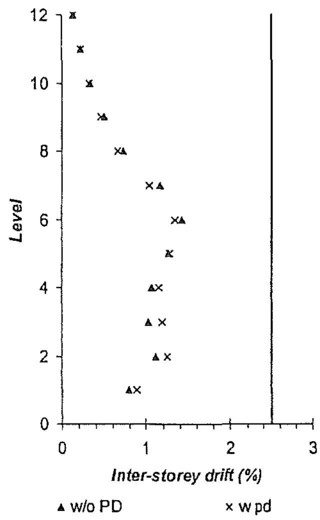



| | *12A/6/E/T* Maximum displacements (m) | | *12A/6/3101/T* Maximum displacements (m) | | *12A/6/1170/T* Maximum displacements (m) | |
|---|---|---|---|---|---|---|
| *Level* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* |
| *1* | 0.026 | 0.032 | 0.026 | 0.032 | 0.026 | 0.030 |
| *2* | 0.061 | 0.070 | 0.061 | 0.070 | 0.067 | 0.080 |
| *3* | 0.094 | 0.110 | 0.094 | 0.110 | 0.109 | 0.132 |
| *4* | 0.133 | 0.152 | 0.133 | 0.152 | 0.149 | 0.177 |
| *5* | 0.169 | 0.188 | 0.169 | 0.188 | 0.185 | 0.212 |
| *6* | 0.200 | 0.217 | 0.200 | 0.217 | 0.215 | 0.238 |
| *7* | 0.224 | 0.239 | 0.224 | 0.239 | 0.233 | 0.252 |
| *8* | 0.240 | 0.253 | 0.240 | 0.253 | 0.240 | 0.258 |
| *9* | 0.249 | 0.261 | 0.249 | 0.261 | 0.244 | 0.264 |
| *10* | 0.253 | 0.266 | 0.253 | 0.266 | 0.248 | 0.268 |
| *11* | 0.254 | 0.267 | 0.254 | 0.267 | 0.249 | 0.269 |
| *12* | 0.253 | 0.267 | 0.253 | 0.267 | 0.249 | 0.268 |

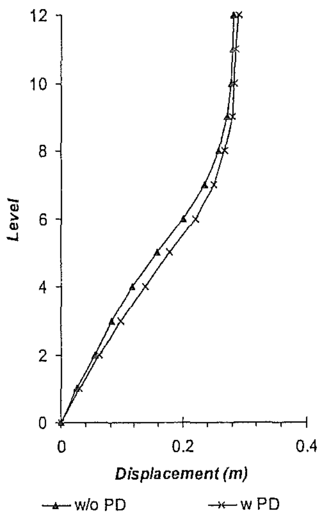
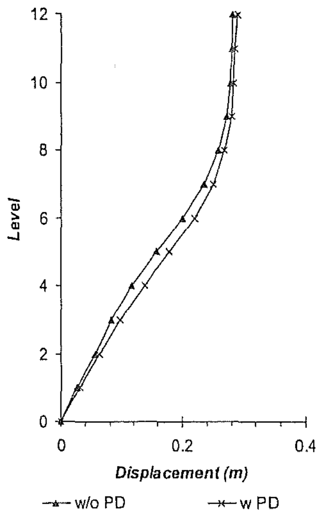
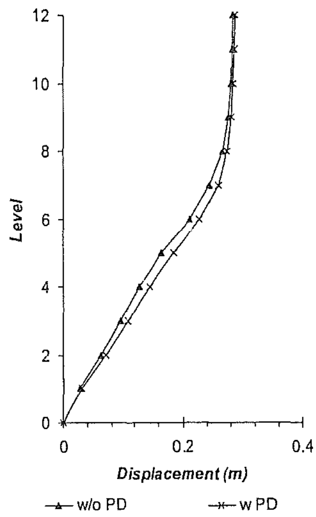
| *Level* | *12A/6/E/T* Central span beam rotations (rad) | *12A/6/3101/T* Central span beam rotations (rad) | *12A/6/1170/T* Central span beam rotations (rad) |
|---|---|---|---|
| *1* | 0.0091 | 0.0091 | 0.0077 |
| *2* | 0.0111 | 0.0112 | 0.0110 |
| *3* | 0.0116 | 0.0116 | 0.0112 |
| *4* | 0.0115 | 0.0115 | 0.0085 |
| *5* | 0.0103 | 0.0103 | 0.0070 |
| *6* | 0.0075 | 0.0075 | 0.0058 |
| *7* | 0.0048 | 0.0048 | 0.0030 |
| *8* | 0.0021 | 0.0021 | 0.0014 |
| *9* | 0.0021 | 0.0021 | 0.0025 |
| *10* | 0.0021 | 0.0021 | 0.0022 |
| *11* | 0.0006 | 0.0006 | 0.0005 |
| *12* | 0.0003 | 0.0003 | 0.0002 |

| El Centro 2 | | | |
|---|---|---|---|
| | *12A/6/E/T* | *12A/6/3101/T* | *12A/6/1170/T* |
| | ***Interior column rotation (rad)*** | ***Interior column rotation (rad)*** | ***Interior column rotation (rad)*** |
| *G.L.* | 0.0055 | 0.0055 | 0.0058 |
| ***1 Below*** | | 0.0008 | 0.0011 |
| ***1 Above*** | | 0.0014 | 0.0072 |
| ***2 Below*** | | 0.0012 | 0.0051 |
| ***2 Above*** | | 0.0011 | 0.0052 |
| ***3 Below*** | | 0.0013 | 0.0056 |
| ***3 Above*** | | 0.0012 | 0.0042 |
| ***4 Below*** | | 0.0012 | 0.0068 |
| ***4 Above*** | | 0.0012 | 0.0043 |
| ***5 Below*** | | 0.0013 | 0.0048 |
| ***5 Above*** | | 0.0010 | 0.0025 |
| ***6 Below*** | | 0.0013 | 0.0051 |
| ***6 Above*** | *Elastic columns at upper levels* | 0.0007 | 0.0012 |
| ***7 Below*** | | 0.0013 | 0.0030 |
| ***7 Above*** | | 0.0008 | 0.0013 |
| ***8 Below*** | | 0.0011 | 0.0017 |
| ***8 Above*** | | 0.0009 | 0.0017 |
| ***9 Below*** | | 0.0008 | 0.0008 |
| ***9 Above*** | | 0.0008 | 0.0008 |
| ***10 Below*** | | 0.0008 | 0.0009 |
| ***10 Above*** | | 0.0006 | 0.0006 |
| ***11 Below*** | | 0.0009 | 0.0010 |
| ***11 Above*** | | 0.0005 | 0.0005 |
| ***12 Below*** | | 0.0006 | 0.0006 |

**Table B-15 Result comparison between models 12A/6/E/T, 12A/6/3101/T and 12A/6/1170/T for Tabas 1 ground acceleration record**

| *Tabas 1* | | | | | | |
|---|---|---|---|---|---|---|
| | *12A/6/E/T* | | *12A/6/3101/T* | | *12A/6/1170/T* | |
| | *Maximum base shear (kN)* | | *Maximum base shear (kN)* | | *Maximum base shear (kN)* | |
| *W/o P-Δ* | 1941.8 | | 1941.8 | | 1944.6 | |
| *With P-Δ* | 2153.5 | | 2152.8 | | 2107.1 | |
| *Inter-storey drifts* |  | |  | |  | |
| | *Maximum inter-storey drifts (% of storey height)* | | *Maximum inter-storey drifts (% of storey height)* | | *Maximum inter-storey drifts (% of storey height)* | |
| *Storey* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* |
| *1* | 0.77 | 0.88 | 0.766 | 0.88 | 0.79 | 0.88 |
| *2* | 0.89 | 1.01 | 0.89 | 1.01 | 1.11 | 1.26 |
| *3* | 0.95 | 1.13 | 0.95 | 1.13 | 1.03 | 1.19 |
| *4* | 1.08 | 1.16 | 1.08 | 1.16 | 1.07 | 1.15 |
| *5* | 1.24 | 1.28 | 1.24 | 1.28 | 1.26 | 1.28 |
| *6* | 1.40 | 1.34 | 1.40 | 1.34 | 1.43 | 1.34 |
| *7* | 1.17 | 1.03 | 1.17 | 1.03 | 1.17 | 1.05 |
| *8* | 0.73 | 0.65 | 0.73 | 0.65 | 0.72 | 0.65 |
| *9* | 0.48 | 0.47 | 0.48 | 0.47 | 0.50 | 0.47 |
| *10* | 0.38 | 0.34 | 0.38 | 0.34 | 0.33 | 0.31 |
| *11* | 0.22 | 0.21 | 0.22 | 0.21 | 0.22 | 0.22 |
| *12* | 0.14 | 0.13 | 0.14 | 0.13 | 0.14 | 0.13 |

*Tabas 1*

| | *12A/6/E/T* | *12A/6/3101/T* | *12A/6/1170/T* |
|---|---|---|---|
| *Maximum displacements* |  |  |  |

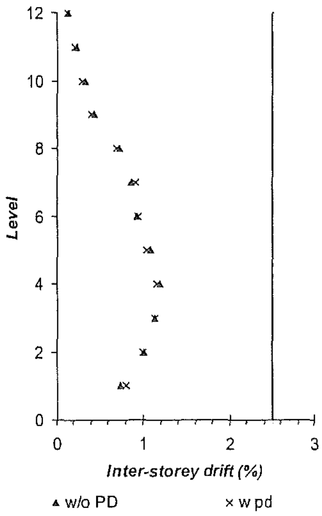
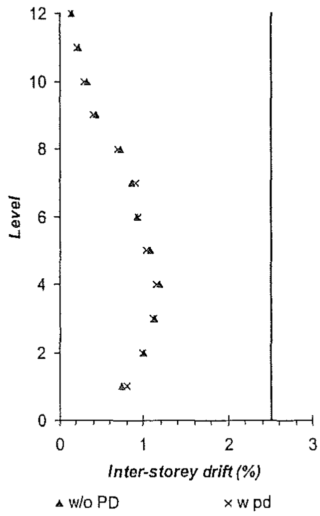


| | *Maximum displacements (m)* | | *Maximum displacements (m)* | | *Maximum displacements (m)* | |
|---|---|---|---|---|---|---|
| *Level* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* |
| *1* | 0.026 | 0.030 | 0.026 | 0.030 | 0.027 | 0.030 |
| *2* | 0.055 | 0.062 | 0.055 | 0.062 | 0.063 | 0.070 |
| *3* | 0.083 | 0.098 | 0.083 | 0.098 | 0.095 | 0.109 |
| *4* | 0.116 | 0.137 | 0.116 | 0.137 | 0.127 | 0.145 |
| *5* | 0.158 | 0.177 | 0.158 | 0.177 | 0.164 | 0.184 |
| *6* | 0.201 | 0.219 | 0.201 | 0.219 | 0.210 | 0.228 |
| *7* | 0.235 | 0.251 | 0.235 | 0.251 | 0.245 | 0.259 |
| *8* | 0.258 | 0.269 | 0.258 | 0.269 | 0.265 | 0.275 |
| *9* | 0.273 | 0.280 | 0.273 | 0.280 | 0.276 | 0.281 |
| *10* | 0.280 | 0.284 | 0.280 | 0.284 | 0.280 | 0.283 |
| *11* | 0.281 | 0.286 | 0.281 | 0.286 | 0.283 | 0.286 |
| *12* | 0.282 | 0.290 | 0.282 | 0.290 | 0.284 | 0.288 |

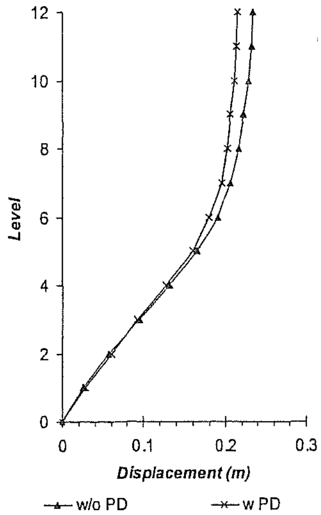
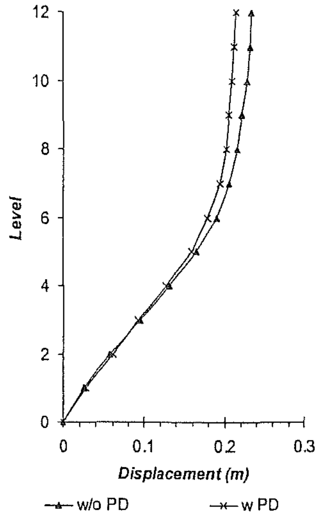
| *Level* | *Central span beam rotations (rad)* | *Central span beam rotations (rad)* | *Central span beam rotations (rad)* |
|---|---|---|---|
| *1* | 0.0080 | 0.0080 | 0.0075 |
| *2* | 0.0095 | 0.0095 | 0.0078 |
| *3* | 0.0100 | 0.0100 | 0.0075 |
| *4* | 0.0114 | 0.0114 | 0.0071 |
| *5* | 0.0130 | 0.0130 | 0.0088 |
| *6* | 0.0118 | 0.0118 | 0.0090 |
| *7* | 0.0074 | 0.0074 | 0.0057 |
| *8* | 0.0042 | 0.0042 | 0.0038 |
| *9* | 0.0028 | 0.0029 | 0.0020 |
| *10* | 0.0006 | 0.0006 | 0.0007 |
| *11* | 0.0004 | 0.0004 | 0.0004 |
| *12* | 0.0002 | 0.0002 | 0.0002 |

| *Tabas 1* | | | |
|---|---|---|---|
| | *12A/6/E/T* | *12A/6/3101/T* | *12A/6/1170/T* |
| | ***Interior column rotation (rad)*** | ***Interior column rotation (rad)*** | ***Interior column rotation (rad)*** |
| *G.L.* | 0.0063 | 0.0063 | 0.0067 |
| ***1 Below*** | *Elastic columns at upper levels* | 0.0010 | 0.0010 |
| ***1 Above*** | | 0.0013 | 0.0056 |
| ***2 Below*** | | 0.0014 | 0.0050 |
| ***2 Above*** | | 0.0011 | 0.0040 |
| ***3 Below*** | | 0.0014 | 0.0046 |
| ***3 Above*** | | 0.0012 | 0.0040 |
| ***4 Below*** | | 0.0012 | 0.0047 |
| ***4 Above*** | | 0.0011 | 0.0062 |
| ***5 Below*** | | 0.0011 | 0.0052 |
| ***5 Above*** | | 0.0009 | 0.0055 |
| ***6 Below*** | | 0.0012 | 0.0050 |
| ***6 Above*** | | 0.0010 | 0.0023 |
| ***7 Below*** | | 0.0015 | 0.0049 |
| ***7 Above*** | | 0.0009 | 0.0023 |
| ***8 Below*** | | 0.0011 | 0.0030 |
| ***8 Above*** | | 0.0009 | 0.0013 |
| ***9 Below*** | | 0.0010 | 0.0020 |
| ***9 Above*** | | 0.0007 | 0.0008 |
| ***10 Below*** | | 0.0010 | 0.0009 |
| ***10 Above*** | | 0.0006 | 0.0006 |
| ***11 Below*** | | 0.0007 | 0.0007 |
| ***11 Above*** | | 0.0004 | 0.0004 |
| ***12 Below*** | | 0.0005 | 0.0005 |

**Table B-16 Result comparison between models 12A/6/E/T and 12A/6/3101/T for Tabas 2 ground acceleration record**

| *Tabas 2* | | | | |
|---|---|---|---|---|
| | *12A/6/E/T* | | *12A/6/3101/T* | |
| | *Maximum base shear (kN)* | | *Maximum base shear (kN)* | |
| *W/o P-Δ* | 1965.2 | | 1965.2 | |
| *With P-Δ* | 2082.7 | | 2086.3 | |
| *Inter-storey drifts* |  | |  | |
| | *Maximum inter-storey drifts (% of storey height)* | | *Maximum inter-storey drifts (% of storey height)* | |
| *Storey* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* |
| *1* | 0.74 | 0.80 | 0.736 | 0.80 |
| *2* | 1.01 | 0.99 | 1.01 | 0.99 |
| *3* | 1.13 | 1.12 | 1.13 | 1.12 |
| *4* | 1.19 | 1.15 | 1.19 | 1.15 |
| *5* | 1.08 | 1.03 | 1.08 | 1.03 |
| *6* | 0.92 | 0.94 | 0.92 | 0.94 |
| *7* | 0.86 | 0.91 | 0.86 | 0.91 |
| *8* | 0.72 | 0.70 | 0.72 | 0.69 |
| *9* | 0.44 | 0.40 | 0.44 | 0.40 |
| *10* | 0.33 | 0.29 | 0.33 | 0.29 |
| *11* | 0.23 | 0.20 | 0.23 | 0.21 |
| *12* | 0.13 | 0.12 | 0.13 | 0.12 |

**Tabas 2**

| | 12A/6/E/T | 12A/6/3101/T |
|---|---|---|
| *Maximum displacements* |  |  |

| | *Maximum displacements (m)* | | *Maximum displacements (m)* | |
|---|---|---|---|---|
| *Level* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* |
| *1* | 0.025 | 0.027 | 0.025 | 0.027 |
| *2* | 0.056 | 0.061 | 0.056 | 0.061 |
| *3* | 0.095 | 0.092 | 0.095 | 0.092 |
| *4* | 0.131 | 0.127 | 0.131 | 0.127 |
| *5* | 0.165 | 0.160 | 0.166 | 0.159 |
| *6* | 0.191 | 0.179 | 0.191 | 0.179 |
| *7* | 0.206 | 0.195 | 0.206 | 0.195 |
| *8* | 0.216 | 0.202 | 0.216 | 0.202 |
| *9* | 0.222 | 0.206 | 0.222 | 0.206 |
| *10* | 0.228 | 0.210 | 0.228 | 0.210 |
| *11* | 0.232 | 0.213 | 0.232 | 0.212 |
| *12* | 0.234 | 0.214 | 0.234 | 0.214 |

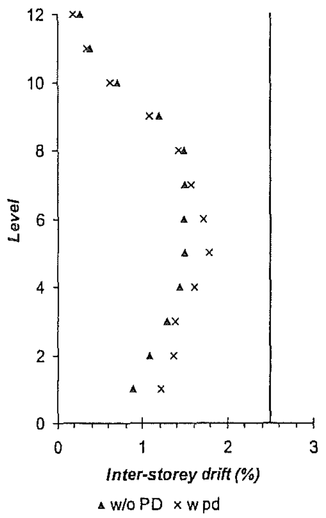
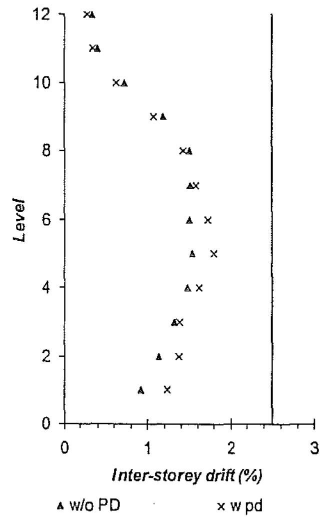
| *Level* | *Central span beam rotations (rad)* | *Central span beam rotations (rad)* |
|---|---|---|
| *1* | 0.0077 | 0.0076 |
| *2* | 0.0093 | 0.0093 |
| *3* | 0.0104 | 0.0104 |
| *4* | 0.0099 | 0.0099 |
| *5* | 0.0084 | 0.0084 |
| *6* | 0.0084 | 0.0085 |
| *7* | 0.0072 | 0.0072 |
| *8* | 0.0040 | 0.0040 |
| *9* | 0.0016 | 0.0016 |
| *10* | 0.0006 | 0.0006 |
| *11* | 0.0004 | 0.0004 |
| *12* | 0.0002 | 0.0002 |

**Tabas 2**

| | *12A/6/E/T* | *12A/6/3101/T* |
|---|---|---|
| | ***Interior column rotation (rad)*** | ***Interior column rotation (rad)*** |
| *G.L.* | 0.0049 | 0.0049 |
| ***1 Below*** | *Elastic columns at upper levels* | 0.0009 |
| ***1 Above*** | | 0.0016 |
| ***2 Below*** | | 0.0013 |
| ***2 Above*** | | 0.0011 |
| ***3 Below*** | | 0.0012 |
| ***3 Above*** | | 0.0010 |
| ***4 Below*** | | 0.0012 |
| ***4 Above*** | | 0.0012 |
| ***5 Below*** | | 0.0013 |
| ***5 Above*** | | 0.0010 |
| ***6 Below*** | | 0.0012 |
| ***6 Above*** | | 0.0008 |
| ***7 Below*** | | 0.0011 |
| ***7 Above*** | | 0.0008 |
| ***8 Below*** | | 0.0012 |
| ***8 Above*** | | 0.0008 |
| ***9 Below*** | | 0.0010 |
| ***9 Above*** | | 0.0007 |
| ***10 Below*** | | 0.0009 |
| ***10 Above*** | | 0.0006 |
| ***11 Below*** | | 0.0007 |
| ***11 Above*** | | 0.0004 |
| ***12 Below*** | | 0.0005 |

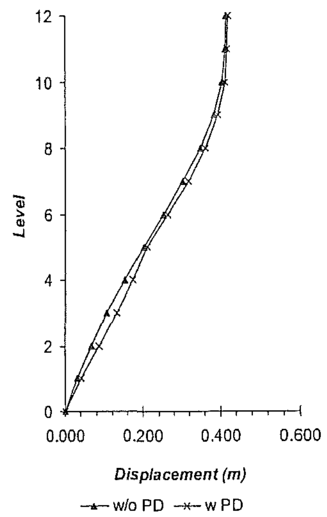
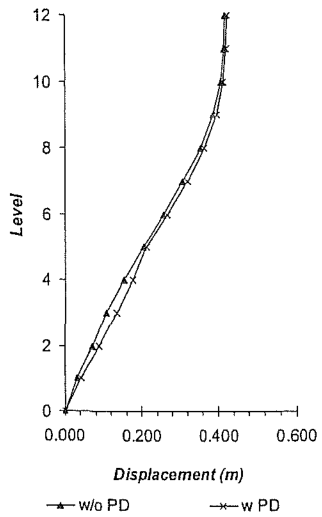
### *B.3 Twelve Storey B Building*

**Table B-17 Results comparison between models 12B/6/E/T, 12B/6/3101/T and 12B/6/1170/T for Modified El Centro NS ground acceleration record**

| *Record "Modified El Centro NS"* | | | | |
|---|---|---|---|---|
| | *12B/6/E/T* | | *12B/6/3101/T* | |
| | *Maximum base shear (kN)* | | *Maximum base shear (kN)* | |
| *W/o P-Δ* | 1850.3 | | 1833.6 | |
| *With P-Δ* | 2012.7 | | 2044.5 | |
| *Inter-storey drifts* |   | |   | |
| | *Maximum inter-storey drifts (% of storey height)* | | *Maximum inter-storey drifts (% of storey height)* | |
| *Storey* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* |
| *1* | 0.89 | 1.21 | 0.92 | 1.23 |
| *2* | 1.08 | 1.36 | 1.13 | 1.38 |
| *3* | 1.29 | 1.38 | 1.33 | 1.39 |
| *4* | 1.43 | 1.60 | 1.46 | 1.62 |
| *5* | 1.49 | 1.78 | 1.53 | 1.79 |
| *6* | 1.48 | 1.71 | 1.50 | 1.73 |
| *7* | 1.49 | 1.57 | 1.51 | 1.58 |
| *8* | 1.49 | 1.43 | 1.50 | 1.43 |
| *9* | 1.19 | 1.08 | 1.19 | 1.07 |
| *10* | 0.71 | 0.61 | 0.71 | 0.62 |
| *11* | 0.37 | 0.34 | 0.40 | 0.34 |
| *12* | 0.26 | 0.18 | 0.34 | 0.28 |

**Record "Modified El Centro NS"**

*Maximum displacements*

| *12B/6/E/T* | *12B/6/3101/T* |
|---|---|
|  |  |

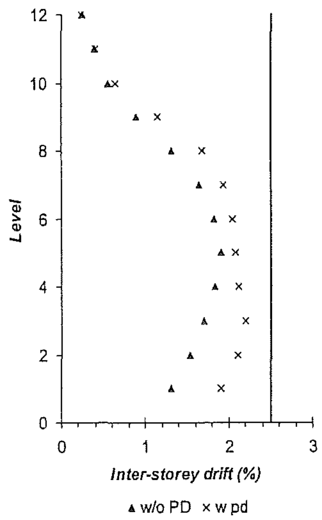
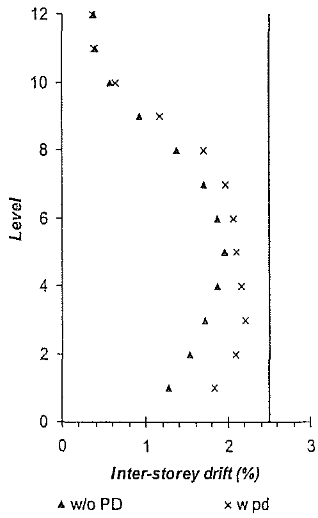


| Level | Maximum displacements (m) Without P-Δ | With P-Δ | Maximum displacements (m) Without P-Δ | With P-Δ |
|---|---|---|---|---|
| 1 | 0.030 | 0.041 | 0.031 | 0.042 |
| 2 | 0.067 | 0.087 | 0.070 | 0.089 |
| 3 | 0.104 | 0.132 | 0.108 | 0.133 |
| 4 | 0.151 | 0.173 | 0.153 | 0.175 |
| 5 | 0.202 | 0.210 | 0.205 | 0.212 |
| 6 | 0.252 | 0.263 | 0.256 | 0.266 |
| 7 | 0.303 | 0.316 | 0.307 | 0.319 |
| 8 | 0.349 | 0.362 | 0.354 | 0.365 |
| 9 | 0.383 | 0.394 | 0.388 | 0.397 |
| 10 | 0.403 | 0.411 | 0.408 | 0.414 |
| 11 | 0.411 | 0.417 | 0.416 | 0.421 |
| 12 | 0.413 | 0.419 | 0.421 | 0.424 |

| Level | Centre span beam rotations (rad) | Centre span beam rotations (rad) |
|---|---|---|
| 1 | 0.0120 | 0.0123 |
| 2 | 0.0129 | 0.0131 |
| 3 | 0.0143 | 0.0145 |
| 4 | 0.0171 | 0.0172 |
| 5 | 0.0179 | 0.0181 |
| 6 | 0.0159 | 0.0160 |
| 7 | 0.0141 | 0.0141 |
| 8 | 0.0119 | 0.0119 |
| 9 | 0.0067 | 0.0067 |
| 10 | 0.0024 | 0.0025 |
| 11 | 0.0005 | 0.0009 |
| 12 | 0.0002 | 0.0001 |

| *Record "Modified El Centro NS"* | | |
|---|---|---|
| | *12B/6/E/T* | *12B/6/3101/T* |
| | *Interior column rotation (rad)* | *Interior column rotation (rad)* |
| *G.L.* | 0.0089 | 0.0090 |
| *1 Below* | | 0.0007 |
| *1 Above* | | 0.0023 |
| *2 Below* | | 0.0013 |
| *2 Above* | | 0.0014 |
| *3 Below* | | 0.0011 |
| *3 Above* | | 0.0011 |
| *4 Below* | | 0.0011 |
| *4 Above* | | 0.0010 |
| *5 Below* | | 0.0010 |
| *5 Above* | | 0.0010 |
| *6 Below* | | 0.0011 |
| *6 Above* | *Elastic columns at upper levels* | 0.0012 |
| *7 Below* | | 0.0010 |
| *7 Above* | | 0.0008 |
| *8 Below* | | 0.0011 |
| *8 Above* | | 0.0007 |
| *9 Below* | | 0.0013 |
| *9 Above* | | 0.0006 |
| *10 Below* | | 0.0011 |
| *10 Above* | | 0.0006 |
| *11 Below* | | 0.0008 |
| *11 Above* | | 0.0005 |
| *12 Below* | | 0.0027 |

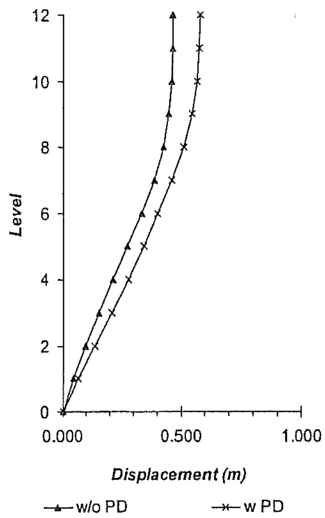
**Table B-18 Results comparison between models 12B/6/E/T, 12B/6/3101/T and 12B/6/1170/T for Modified El Centro EW ground acceleration record**

| *Record "Modified El Centro EW"* | | | | |
|---|---|---|---|---|
| | *12B/6/E/T* | | *12B/6/3101/T* | |
| | *Maximum base shear (kN)* | | *Maximum base shear (kN)* | |
| *W/o P-Δ* | 1689.4 | | 1704.9 | |
| *With P-Δ* | 2133.9 | | 2107.9 | |
| *Inter-storey drifts* |  | |  | |
| | *Maximum inter-storey drifts (% of storey height)* | | *Maximum inter-storey drifts (% of storey height)* | |
| *Storey* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* |
| *1* | 1.32 | 1.91 | 1.27 | 1.83 |
| *2* | 1.54 | 2.11 | 1.52 | 2.09 |
| *3* | 1.70 | 2.20 | 1.71 | 2.21 |
| *4* | 1.84 | 2.11 | 1.87 | 2.16 |
| *5* | 1.91 | 2.08 | 1.95 | 2.10 |
| *6* | 1.82 | 2.04 | 1.87 | 2.06 |
| *7* | 1.64 | 1.94 | 1.70 | 1.97 |
| *8* | 1.32 | 1.68 | 1.38 | 1.71 |
| *9* | 0.89 | 1.14 | 0.92 | 1.16 |
| *10* | 0.55 | 0.63 | 0.57 | 0.63 |
| *11* | 0.39 | 0.40 | 0.39 | 0.38 |
| *12* | 0.24 | 0.23 | 0.37 | 0.35 |

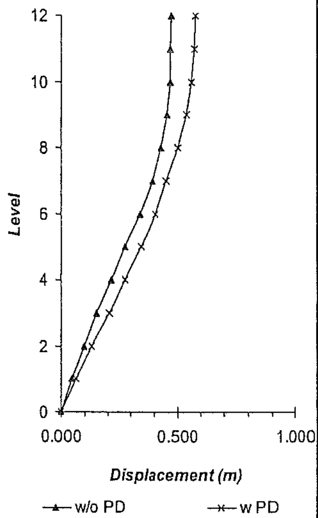
**Record "Modified El Centro EW"**

*Maximum displacements*

**12B/6/E/T**




**12B/6/3101/T**

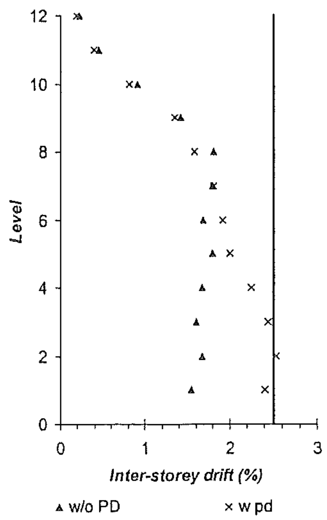
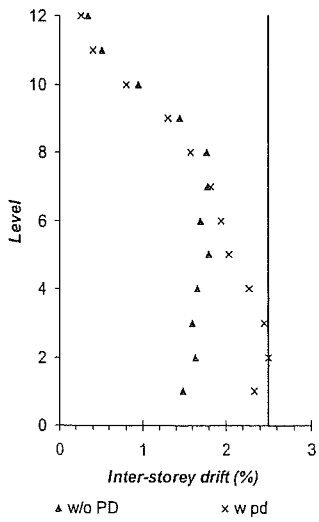



| | 12B/6/E/T Maximum displacements (m) | | 12B/6/3101/T Maximum displacements (m) | |
|---|---|---|---|---|
| *Level* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* |
| 1 | 0.045 | 0.065 | 0.043 | 0.062 |
| 2 | 0.096 | 0.137 | 0.095 | 0.133 |
| 3 | 0.153 | 0.208 | 0.151 | 0.205 |
| 4 | 0.213 | 0.279 | 0.212 | 0.277 |
| 5 | 0.276 | 0.344 | 0.277 | 0.345 |
| 6 | 0.336 | 0.401 | 0.339 | 0.405 |
| 7 | 0.390 | 0.462 | 0.394 | 0.455 |
| 8 | 0.427 | 0.510 | 0.432 | 0.504 |
| 9 | 0.452 | 0.545 | 0.459 | 0.539 |
| 10 | 0.463 | 0.566 | 0.470 | 0.560 |
| 11 | 0.467 | 0.577 | 0.474 | 0.571 |
| 12 | 0.468 | 0.583 | 0.475 | 0.578 |

| *Level* | *Centre span beam rotations (rad)* (12B/6/E/T) | *Centre span beam rotations (rad)* (12B/6/3101/T) |
|---|---|---|
| 1 | 0.02006 | 0.0194 |
| 2 | 0.02172 | 0.0216 |
| 3 | 0.02175 | 0.0222 |
| 4 | 0.02115 | 0.0213 |
| 5 | 0.02115 | 0.0214 |
| 6 | 0.02019 | 0.0206 |
| 7 | 0.01789 | 0.0183 |
| 8 | 0.01361 | 0.0137 |
| 9 | 0.00670 | 0.0070 |
| 10 | 0.00272 | 0.0026 |
| 11 | 0.00101 | 0.0015 |
| 12 | 0.00032 | 0.0002 |

| *Record "Modified El Centro EW"* | | |
|---|---|---|
| | *12B/6/E/T* | *12B/6/3101/T* |
| | *Interior column rotation (rad)* | *Interior column rotation (rad)* |
| *G.L.* | 0.01493 | 0.0142 |
| *1 Below* | Elastic columns at upper levels | 0.0007 |
| *1 Above* | | 0.0015 |
| *2 Below* | | 0.0010 |
| *2 Above* | | 0.0012 |
| *3 Below* | | 0.0011 |
| *3 Above* | | 0.0011 |
| *4 Below* | | 0.0011 |
| *4 Above* | | 0.0008 |
| *5 Below* | | 0.0011 |
| *5 Above* | | 0.0007 |
| *6 Below* | | 0.0012 |
| *6 Above* | | 0.0008 |
| *7 Below* | | 0.0014 |
| *7 Above* | | 0.0009 |
| *8 Below* | | 0.0014 |
| *8 Above* | | 0.0008 |
| *9 Below* | | 0.0014 |
| *9 Above* | | 0.0007 |
| *10 Below* | | 0.0011 |
| *10 Above* | | 0.0006 |
| *11 Below* | | 0.0008 |
| *11 Above* | | 0.0006 |
| *12 Below* | | 0.0038 |

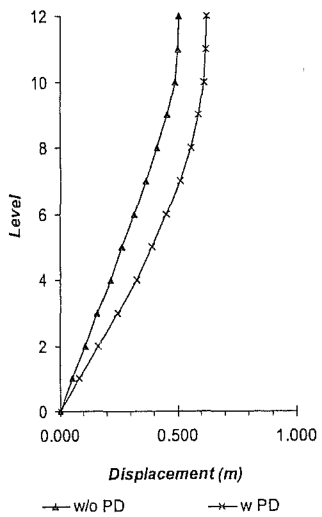
**Table B-19 Results comparison between models 12B/6/E/T, 12B/6/3101/T and 12B/6/1170/T for Modified TAFT 1 ground acceleration record**

| *Record "Modified TAFT 1"* | | | | |
|---|---|---|---|---|
| | *12B/6/E/T* | | *12B/6/3101/T* | |
| | *Maximum base shear (kN)* | | *Maximum base shear (kN)* | |
| *W/o P-Δ* | 1789.9 | | 1817.8 | |
| *With P-Δ* | 2392.3 | | 2361.4 | |
| *Inter-storey drifts* |  | |  | |
| | *Maximum inter-storey drifts (% of storey height)* | | *Maximum inter-storey drifts (% of storey height)* | |
| *Storey* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* |
| *1* | 1.54 | 2.40 | 1.48 | 2.33 |
| *2* | 1.67 | 2.53 | 1.65 | 2.50 |
| *3* | 1.61 | 2.45 | 1.60 | 2.45 |
| *4* | 1.67 | 2.25 | 1.66 | 2.27 |
| *5* | 1.79 | 2.01 | 1.80 | 2.04 |
| *6* | 1.69 | 1.92 | 1.70 | 1.94 |
| *7* | 1.80 | 1.81 | 1.78 | 1.82 |
| *8* | 1.81 | 1.58 | 1.77 | 1.58 |
| *9* | 1.43 | 1.34 | 1.45 | 1.30 |
| *10* | 0.91 | 0.82 | 0.95 | 0.80 |
| *11* | 0.45 | 0.39 | 0.51 | 0.40 |
| *12* | 0.22 | 0.19 | 0.34 | 0.25 |

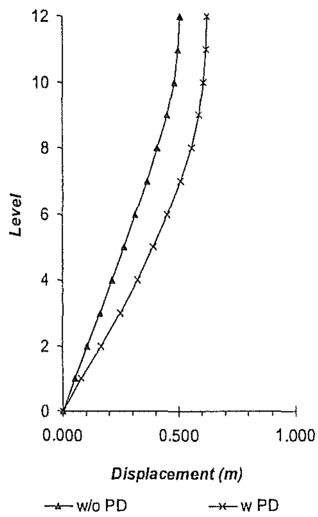
**Record "Modified TAFT 1"**

*Maximum displacements*

**12B/6/E/T**




**12B/6/3101/T**

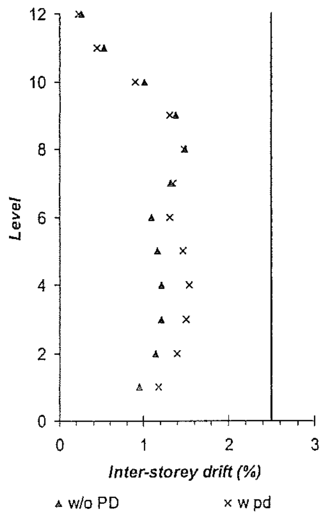
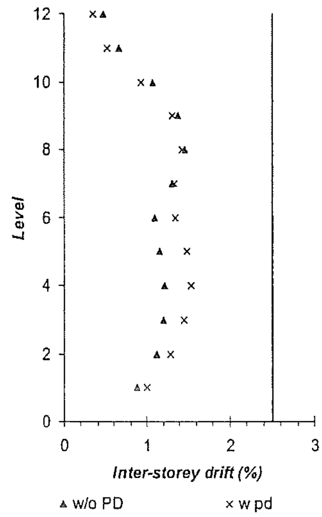



| | 12B/6/E/T | | 12B/6/3101/T | |
|---|---|---|---|---|
| | *Maximum displacements (m)* | | *Maximum displacements (m)* | |
| ***Level*** | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* |
| *1* | 0.052 | 0.082 | 0.050 | 0.079 |
| *2* | 0.109 | 0.167 | 0.106 | 0.164 |
| *3* | 0.164 | 0.250 | 0.161 | 0.247 |
| *4* | 0.217 | 0.327 | 0.213 | 0.324 |
| *5* | 0.269 | 0.395 | 0.265 | 0.394 |
| *6* | 0.319 | 0.456 | 0.314 | 0.456 |
| *7* | 0.368 | 0.511 | 0.365 | 0.511 |
| *8* | 0.415 | 0.557 | 0.412 | 0.558 |
| *9* | 0.457 | 0.591 | 0.452 | 0.590 |
| *10* | 0.487 | 0.611 | 0.483 | 0.609 |
| *11* | 0.502 | 0.619 | 0.500 | 0.618 |
| *12* | 0.508 | 0.623 | 0.510 | 0.624 |

| ***Level*** | ***Centre span beam rotations (rad)*** | ***Centre span beam rotations (rad)*** |
|---|---|---|
| *1* | 0.0250 | 0.0247 |
| *2* | 0.0255 | 0.0254 |
| *3* | 0.0239 | 0.0239 |
| *4* | 0.0216 | 0.0218 |
| *5* | 0.0198 | 0.0201 |
| *6* | 0.0188 | 0.0189 |
| *7* | 0.0164 | 0.0165 |
| *8* | 0.0145 | 0.0139 |
| *9* | 0.0097 | 0.0095 |
| *10* | 0.0037 | 0.0034 |
| *11* | 0.0006 | 0.0009 |
| *12* | 0.0003 | 0.0002 |

| Record "Modified TAFT 1" | | |
|---|---|---|
| | *12B/6/E/T* | *12B/6/3101/T* |
| | *Interior column rotation (rad)* | *Interior column rotation (rad)* |
| *G.L.* | 0.01844 | 0.0178 |
| *1 Below* | | 0.0007 |
| *1 Above* | | 0.0013 |
| *2 Below* | | 0.0013 |
| *2 Above* | | 0.0010 |
| *3 Below* | | 0.0012 |
| *3 Above* | | 0.0012 |
| *4 Below* | | 0.0012 |
| *4 Above* | | 0.0008 |
| *5 Below* | | 0.0011 |
| *5 Above* | | 0.0007 |
| *6 Below* | | 0.0012 |
| *6 Above* | *Elastic columns at upper levels* | 0.0011 |
| *7 Below* | | 0.0014 |
| *7 Above* | | 0.0010 |
| *8 Below* | | 0.0012 |
| *8 Above* | | 0.0007 |
| *9 Below* | | 0.0012 |
| *9 Above* | | 0.0006 |
| *10 Below* | | 0.0013 |
| *10 Above* | | 0.0005 |
| *11 Below* | | 0.0008 |
| *11 Above* | | 0.0005 |
| *12 Below* | | 0.0025 |

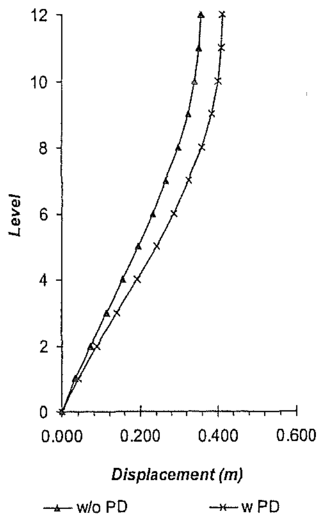
**Table B-20 Results comparison between models 12B/6/E/T, 12B/6/3101/T and 12B/6/1170/T for Modified TAFT 2 ground acceleration record**

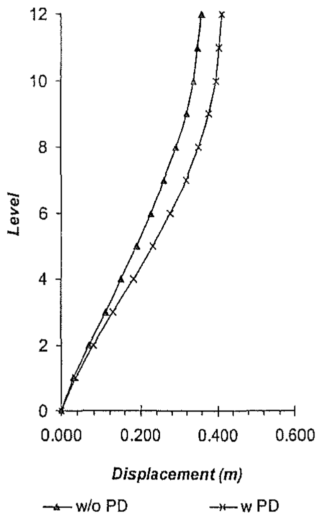
| *Record "Modified TAFT 2"* | | | | |
|---|---|---|---|---|
| | *12B/6/E/T* | | *12B/6/3101/T* | |
| | *Maximum base shear (kN)* | | *Maximum base shear (kN)* | |
| *W/o P-Δ* | 1688.5 | | 1742.2 | |
| *With P-Δ* | 1832.0 | | 1815.4 | |
| *Inter-storey drifts* |   | |   | |
| | *Maximum inter-storey drifts (% of storey height)* | | *Maximum inter-storey drifts (% of storey height)* | |
| *Storey* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* |
| *1* | 0.95 | 1.17 | 0.88 | 1.00 |
| *2* | 1.15 | 1.39 | 1.12 | 1.29 |
| *3* | 1.22 | 1.51 | 1.21 | 1.46 |
| *4* | 1.21 | 1.54 | 1.21 | 1.54 |
| *5* | 1.16 | 1.47 | 1.16 | 1.49 |
| *6* | 1.09 | 1.31 | 1.10 | 1.34 |
| *7* | 1.33 | 1.35 | 1.31 | 1.34 |
| *8* | 1.49 | 1.48 | 1.47 | 1.44 |
| *9* | 1.39 | 1.31 | 1.38 | 1.31 |
| *10* | 1.01 | 0.90 | 1.07 | 0.92 |
| *11* | 0.52 | 0.44 | 0.66 | 0.51 |
| *12* | 0.26 | 0.22 | 0.46 | 0.34 |

| *Record "Modified TAFT 2"* |
|---|

| | 12B/6/E/T | 12B/6/3101/T |
|---|---|---|

*Maximum displacements*




| | *Maximum displacements (m)* | | *Maximum displacements (m)* | |
|---|---|---|---|---|
| *Level* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* |
| *1* | 0.032 | 0.040 | 0.030 | 0.034 |
| *2* | 0.071 | 0.087 | 0.068 | 0.078 |
| *3* | 0.113 | 0.139 | 0.109 | 0.127 |
| *4* | 0.154 | 0.191 | 0.150 | 0.180 |
| *5* | 0.193 | 0.241 | 0.189 | 0.230 |
| *6* | 0.229 | 0.285 | 0.225 | 0.276 |
| *7* | 0.264 | 0.325 | 0.260 | 0.317 |
| *8* | 0.297 | 0.359 | 0.292 | 0.352 |
| *9* | 0.324 | 0.384 | 0.319 | 0.378 |
| *10* | 0.342 | 0.400 | 0.338 | 0.395 |
| *11* | 0.351 | 0.408 | 0.349 | 0.404 |
| *12* | 0.356 | 0.411 | 0.360 | 0.410 |

| *Level* | *Centre span beam rotations (rad)* | *Centre span beam rotations (rad)* |
|---|---|---|
| *1* | 0.0118 | 0.0102 |
| *2* | 0.0138 | 0.0129 |
| *3* | 0.0146 | 0.0144 |
| *4* | 0.0147 | 0.0149 |
| *5* | 0.0135 | 0.0140 |
| *6* | 0.0116 | 0.0120 |
| *7* | 0.0136 | 0.0134 |
| *8* | 0.0137 | 0.0135 |
| *9* | 0.0099 | 0.0103 |
| *10* | 0.0046 | 0.0053 |
| *11* | 0.0010 | 0.0017 |
| *12* | 0.0003 | 0.0001 |

*Record "Modified TAFT 2"*

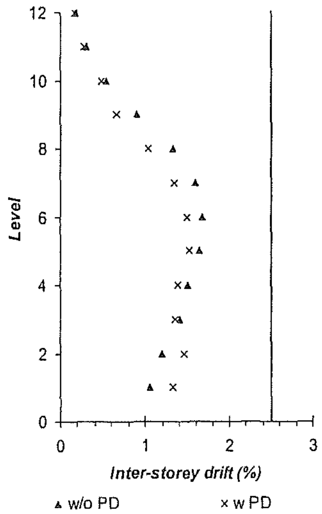
| | *12B/6/E/T* | *12B/6/3101/T* |
|---|---|---|
| | *Interior column rotation (rad)* | *Interior column rotation (rad)* |
| *G.L.* | 0.0083 | 0.0069 |
| *1 Below* | *Elastic columns at upper levels* | 0.0007 |
| *1 Above* | | 0.0014 |
| *2 Below* | | 0.0010 |
| *2 Above* | | 0.0011 |
| *3 Below* | | 0.0010 |
| *3 Above* | | 0.0010 |
| *4 Below* | | 0.0010 |
| *4 Above* | | 0.0011 |
| *5 Below* | | 0.0009 |
| *5 Above* | | 0.0010 |
| *6 Below* | | 0.0010 |
| *6 Above* | | 0.0013 |
| *7 Below* | | 0.0009 |
| *7 Above* | | 0.0011 |
| *8 Below* | | 0.0011 |
| *8 Above* | | 0.0007 |
| *9 Below* | | 0.0012 |
| *9 Above* | | 0.0006 |
| *10 Below* | | 0.0012 |
| *10 Above* | | 0.0005 |
| *11 Below* | | 0.0010 |
| *11 Above* | | 0.0005 |
| *12 Below* | | 0.0037 |

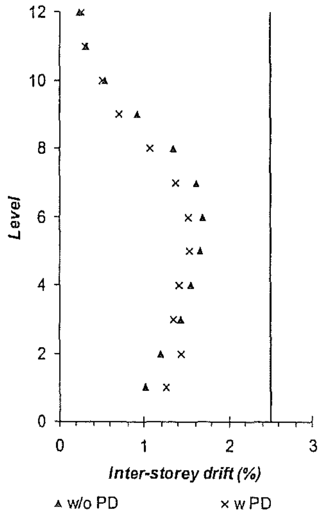
**Table B-21 Results comparison between models 12B/6/E/T, 12B/6/3101/T and 12B/6/1170/T for El Centro 1 ground acceleration record**

*Record "El Centro 1"*

| | *12B/6/E/T* | *12B/6/3101/T* |
|---|---|---|
| | *Maximum base shear (kN)* | *Maximum base shear (kN)* |
| *W/o P-Δ* | 1778.1 | 1789.9 |
| *With P-Δ* | 2067.3 | 2061.1 |

*Inter-storey drifts*



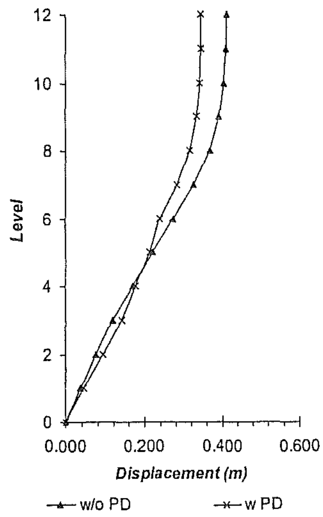

| | *12B/6/E/T* | | *12B/6/3101/T* | |
|---|---|---|---|---|
| | *Maximum inter-storey drifts (% of storey height)* | | *Maximum inter-storey drifts (% of storey height)* | |
| *Storey* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* |
| *1* | 1.05 | 1.32 | 1.02 | 1.26 |
| *2* | 1.20 | 1.46 | 1.19 | 1.44 |
| *3* | 1.41 | 1.35 | 1.44 | 1.35 |
| *4* | 1.51 | 1.39 | 1.55 | 1.41 |
| *5* | 1.64 | 1.51 | 1.67 | 1.54 |
| *6* | 1.68 | 1.49 | 1.70 | 1.52 |
| *7* | 1.59 | 1.34 | 1.61 | 1.38 |
| *8* | 1.32 | 1.04 | 1.35 | 1.06 |
| *9* | 0.90 | 0.66 | 0.92 | 0.70 |
| *10* | 0.53 | 0.47 | 0.54 | 0.50 |
| *11* | 0.31 | 0.29 | 0.31 | 0.31 |
| *12* | 0.18 | 0.17 | 0.24 | 0.25 |

*Record "El Centro 1"*

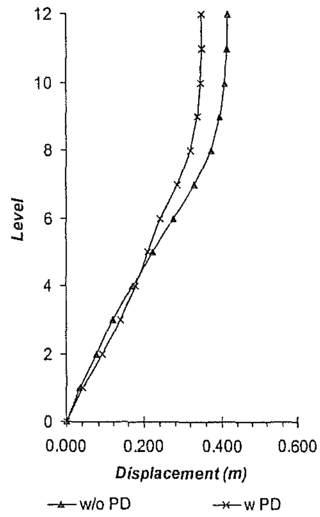
| | *12B/6/E/T* | *12B/6/3101/T* |
|---|---|---|

*Maximum displacements*

*12B/6/E/T*




*12B/6/3101/T*



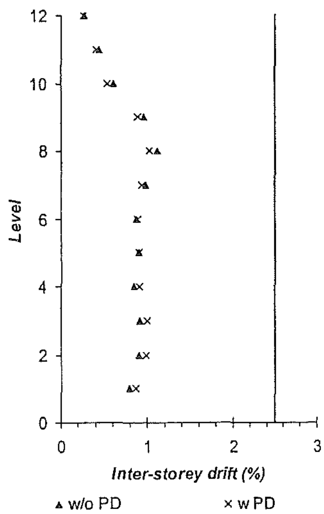
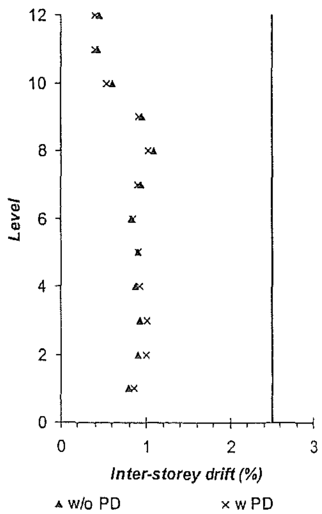

| | *Maximum displacements (m)* | | *Maximum displacements (m)* | |
|---|---|---|---|---|
| *Level* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* |
| *1* | 0.036 | 0.045 | 0.035 | 0.043 |
| *2* | 0.076 | 0.095 | 0.075 | 0.092 |
| *3* | 0.119 | 0.140 | 0.118 | 0.138 |
| *4* | 0.169 | 0.179 | 0.170 | 0.177 |
| *5* | 0.220 | 0.211 | 0.222 | 0.210 |
| *6* | 0.273 | 0.239 | 0.276 | 0.241 |
| *7* | 0.326 | 0.282 | 0.329 | 0.286 |
| *8* | 0.367 | 0.315 | 0.371 | 0.319 |
| *9* | 0.392 | 0.333 | 0.396 | 0.338 |
| *10* | 0.404 | 0.341 | 0.409 | 0.345 |
| *11* | 0.409 | 0.343 | 0.413 | 0.348 |
| *12* | 0.413 | 0.344 | 0.418 | 0.350 |

| ***Level*** | ***Centre span beam rotations (rad)*** | ***Centre span beam rotations (rad)*** |
|---|---|---|
| *1* | 0.0132 | 0.0128 |
| *2* | 0.0134 | 0.0133 |
| *3* | 0.0122 | 0.0125 |
| *4* | 0.0142 | 0.0144 |
| *5* | 0.0152 | 0.0154 |
| *6* | 0.0139 | 0.0142 |
| *7* | 0.0129 | 0.0134 |
| *8* | 0.0081 | 0.0085 |
| *9* | 0.0050 | 0.0050 |
| *10* | 0.0023 | 0.0028 |
| *11* | 0.0006 | 0.0007 |
| *12* | 0.0003 | 0.0002 |

***Record "El Centro 1"***

| | *12B/6/E/T* | *12B/6/3101/T* |
|---|---|---|
| | ***Interior column rotation (rad)*** | ***Interior column rotation (rad)*** |
| *G.L.* | 0.0094 | 0.0088 |
| *1 Below* | *Elastic columns at upper levels* | 0.0007 |
| *1 Above* | | 0.0014 |
| *2 Below* | | 0.0012 |
| *2 Above* | | 0.0012 |
| *3 Below* | | 0.0011 |
| *3 Above* | | 0.0010 |
| *4 Below* | | 0.0012 |
| *4 Above* | | 0.0009 |
| *5 Below* | | 0.0010 |
| *5 Above* | | 0.0007 |
| *6 Below* | | 0.0010 |
| *6 Above* | | 0.0011 |
| *7 Below* | | 0.0013 |
| *7 Above* | | 0.0009 |
| *8 Below* | | 0.0013 |
| *8 Above* | | 0.0006 |
| *9 Below* | | 0.0012 |
| *9 Above* | | 0.0006 |
| *10 Below* | | 0.0010 |
| *10 Above* | | 0.0005 |
| *11 Below* | | 0.0008 |
| *11 Above* | | 0.0006 |
| *12 Below* | | 0.0028 |

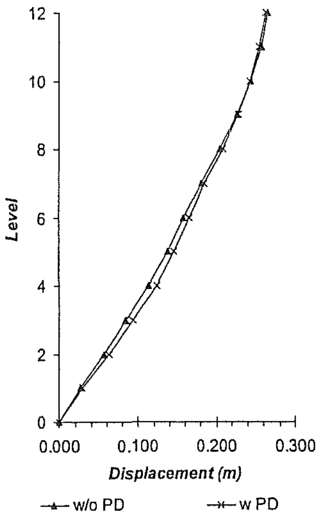
**Table B-22 Results comparison between models 12B/6/E/T, 12B/6/3101/T and 12B/6/1170/T for El Centro 2 ground acceleration record**

| *Record " El Centro 2"* | | | | |
|---|---|---|---|---|
| | *12B/6/E/T* | | *12B/6/3101/T* | |
| | *Maximum base shear (kN)* | | *Maximum base shear (kN)* | |
| *W/o P-Δ* | 1818.6 | | 1848.5 | |
| *With P-Δ* | 2021.9 | | 2042.3 | |
| *Inter-storey drifts* |   | |   | |
| | *Maximum inter-storey drifts (% of storey height)* | | *Maximum inter-storey drifts (% of storey height)* | |
| *Storey* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* |
| *1* | 0.79 | 0.87 | 0.78 | 0.85 |
| *2* | 0.90 | 0.98 | 0.90 | 0.99 |
| *3* | 0.91 | 1.00 | 0.92 | 1.01 |
| *4* | 0.86 | 0.91 | 0.87 | 0.92 |
| *5* | 0.90 | 0.89 | 0.90 | 0.90 |
| *6* | 0.88 | 0.89 | 0.83 | 0.84 |
| *7* | 0.99 | 0.94 | 0.94 | 0.90 |
| *8* | 1.11 | 1.02 | 1.10 | 1.02 |
| *9* | 0.96 | 0.89 | 0.95 | 0.91 |
| *10* | 0.61 | 0.54 | 0.61 | 0.53 |
| *11* | 0.43 | 0.41 | 0.43 | 0.40 |
| *12* | 0.27 | 0.25 | 0.45 | 0.40 |

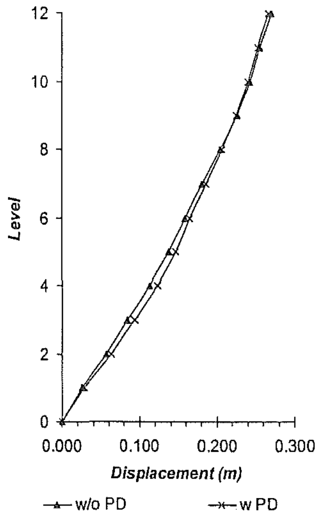
*Record " El Centro 2"*

*Maximum displacements*

*12B/6/E/T*




*12B/6/3101/T*

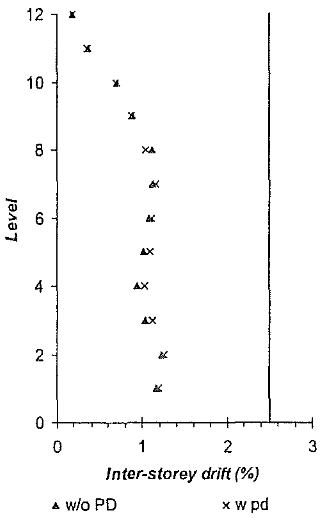
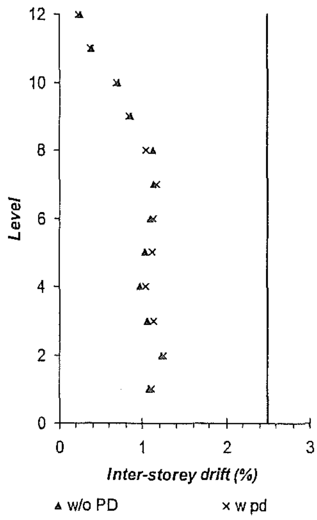



| | *12B/6/E/T* | | *12B/6/3101/T* | |
|---|---|---|---|---|
| | *Maximum displacements (m)* | | *Maximum displacements (m)* | |
| *Level* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* |
| *1* | 0.027 | 0.029 | 0.027 | 0.029 |
| *2* | 0.057 | 0.063 | 0.056 | 0.063 |
| *3* | 0.084 | 0.093 | 0.084 | 0.093 |
| *4* | 0.113 | 0.123 | 0.113 | 0.124 |
| *5* | 0.137 | 0.146 | 0.138 | 0.147 |
| *6* | 0.158 | 0.165 | 0.159 | 0.165 |
| *7* | 0.181 | 0.185 | 0.181 | 0.185 |
| *8* | 0.204 | 0.208 | 0.205 | 0.207 |
| *9* | 0.226 | 0.229 | 0.226 | 0.226 |
| *10* | 0.244 | 0.243 | 0.242 | 0.240 |
| *11* | 0.257 | 0.255 | 0.256 | 0.253 |
| *12* | 0.265 | 0.263 | 0.270 | 0.267 |

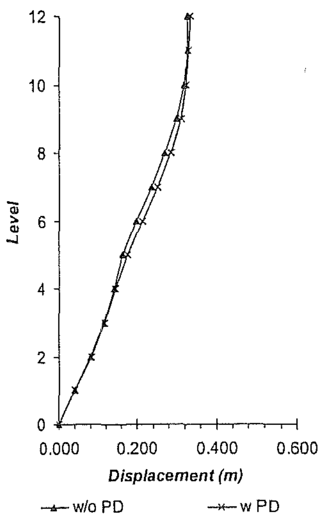
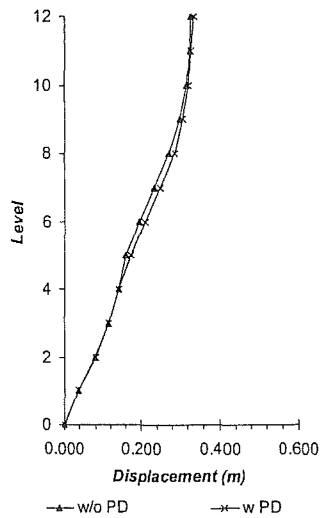
| ***Level*** | ***Centre span beam rotations (rad)*** — 12B/6/E/T | ***Centre span beam rotations (rad)*** — 12B/6/3101/T |
|---|---|---|
| *1* | 0.0082 | 0.0081 |
| *2* | 0.0087 | 0.0089 |
| *3* | 0.0083 | 0.0085 |
| *4* | 0.0080 | 0.0080 |
| *5* | 0.0075 | 0.0075 |
| *6* | 0.0081 | 0.0077 |
| *7* | 0.0096 | 0.0101 |
| *8* | 0.0111 | 0.0112 |
| *9* | 0.0067 | 0.0068 |
| *10* | 0.0028 | 0.0024 |
| *11* | 0.0017 | 0.0025 |
| *12* | 0.0004 | 0.0002 |

| *Record " El Centro 2"* | | |
|---|---|---|
| | *12B/6/E/T* | *12B/6/3101/T* |
| | *Interior column rotation (rad)* | *Interior column rotation (rad)* |
| *G.L.* | 0.0058 | 0.0057 |
| *1 Below* | *Elastic columns at upper levels* | 0.0007 |
| *1 Above* | | 0.0012 |
| *2 Below* | | 0.0013 |
| *2 Above* | | 0.0008 |
| *3 Below* | | 0.0012 |
| *3 Above* | | 0.0009 |
| *4 Below* | | 0.0011 |
| *4 Above* | | 0.0008 |
| *5 Below* | | 0.0010 |
| *5 Above* | | 0.0008 |
| *6 Below* | | 0.0009 |
| *6 Above* | | 0.0013 |
| *7 Below* | | 0.0010 |
| *7 Above* | | 0.0009 |
| *8 Below* | | 0.0010 |
| *8 Above* | | 0.0009 |
| *9 Below* | | 0.0012 |
| *9 Above* | | 0.0007 |
| *10 Below* | | 0.0012 |
| *10 Above* | | 0.0008 |
| *11 Below* | | 0.0008 |
| *11 Above* | | 0.0007 |
| *12 Below* | | 0.0045 |

**Table B-23 Results comparison between models 12B/6/E/T, 12B/6/3101/T and 12B/6/1170/T for Tabas 1 ground acceleration record**

| *Record "Tabas 1"* | | | | |
|---|---|---|---|---|
| | *12B/6/E/T* | | *12B/6/3101/T* | |
| | *Maximum base shear (kN)* | | *Maximum base shear (kN)* | |
| *W/o P-Δ* | 1741.5 | | 1755.9 | |
| *With P-Δ* | 1953.7 | | 1890.7 | |
| *Inter-storey drifts* |  | |  | |
| | *Maximum inter-storey drifts (% of storey height)* | | *Maximum inter-storey drifts (% of storey height)* | |
| *Storey* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* |
| *1* | 1.16 | 1.19 | 1.08 | 1.10 |
| *2* | 1.23 | 1.25 | 1.23 | 1.25 |
| *3* | 1.03 | 1.12 | 1.06 | 1.13 |
| *4* | 0.95 | 1.03 | 0.97 | 1.04 |
| *5* | 1.02 | 1.09 | 1.03 | 1.11 |
| *6* | 1.08 | 1.11 | 1.09 | 1.12 |
| *7* | 1.12 | 1.16 | 1.13 | 1.17 |
| *8* | 1.11 | 1.04 | 1.14 | 1.04 |
| *9* | 0.89 | 0.87 | 0.86 | 0.84 |
| *10* | 0.71 | 0.69 | 0.70 | 0.67 |
| *11* | 0.36 | 0.35 | 0.39 | 0.38 |
| *12* | 0.18 | 0.18 | 0.25 | 0.24 |

**Record "Tabas 1"**

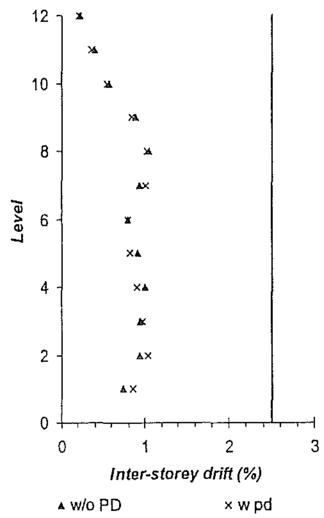
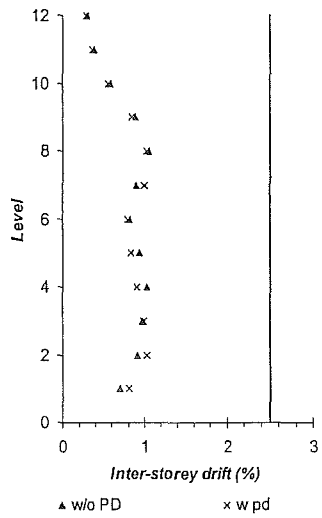
| | 12B/6/E/T | 12B/6/3101/T |
|---|---|---|
| Maximum displacements |  |  |

| | Maximum displacements (m) | | Maximum displacements (m) | |
|---|---|---|---|---|
| *Level* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* |
| 1 | 0.040 | 0.040 | 0.037 | 0.037 |
| 2 | 0.081 | 0.083 | 0.078 | 0.080 |
| 3 | 0.116 | 0.118 | 0.114 | 0.116 |
| 4 | 0.141 | 0.143 | 0.141 | 0.142 |
| 5 | 0.162 | 0.175 | 0.159 | 0.173 |
| 6 | 0.197 | 0.211 | 0.195 | 0.209 |
| 7 | 0.235 | 0.251 | 0.233 | 0.249 |
| 8 | 0.271 | 0.286 | 0.270 | 0.285 |
| 9 | 0.300 | 0.309 | 0.299 | 0.308 |
| 10 | 0.317 | 0.323 | 0.317 | 0.322 |
| 11 | 0.324 | 0.329 | 0.325 | 0.328 |
| 12 | 0.326 | 0.334 | 0.327 | 0.334 |

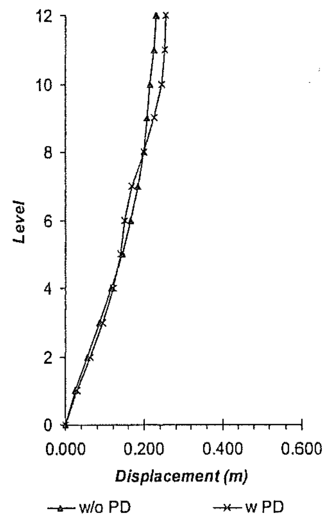
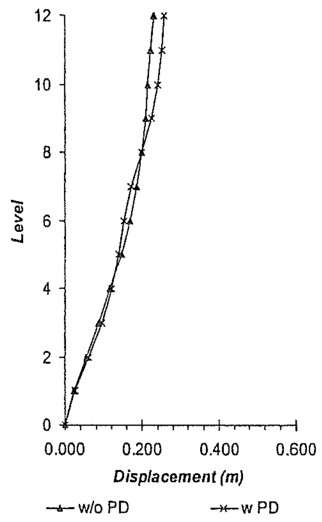
| *Level* | *Centre span beam rotations (rad)* | *Centre span beam rotations (rad)* |
|---|---|---|
| 1 | 0.0116 | 0.0110 |
| 2 | 0.0106 | 0.0110 |
| 3 | 0.0095 | 0.0097 |
| 4 | 0.0095 | 0.0098 |
| 5 | 0.0104 | 0.0106 |
| 6 | 0.0105 | 0.0107 |
| 7 | 0.0120 | 0.0118 |
| 8 | 0.0099 | 0.0097 |
| 9 | 0.0084 | 0.0079 |
| 10 | 0.0038 | 0.0042 |
| 11 | 0.0007 | 0.0007 |
| 12 | 0.0003 | 0.0002 |

| *Record "Tabas 1"* | | |
|---|---|---|
| | *12B/6/E/T* | *12B/6/3101/T* |
| | *Interior column rotation (rad)* | *Interior column rotation (rad)* |
| *G.L.* | 0.0089 | 0.0079 |
| *1 Below* | *Elastic columns at upper levels* | 0.0007 |
| *1 Above* | | 0.0011 |
| *2 Below* | | 0.0011 |
| *2 Above* | | 0.0008 |
| *3 Below* | | 0.0012 |
| *3 Above* | | 0.0008 |
| *4 Below* | | 0.0012 |
| *4 Above* | | 0.0009 |
| *5 Below* | | 0.0010 |
| *5 Above* | | 0.0008 |
| *6 Below* | | 0.0009 |
| *6 Above* | | 0.0012 |
| *7 Below* | | 0.0009 |
| *7 Above* | | 0.0012 |
| *8 Below* | | 0.0011 |
| *8 Above* | | 0.0009 |
| *9 Below* | | 0.0011 |
| *9 Above* | | 0.0007 |
| *10 Below* | | 0.0012 |
| *10 Above* | | 0.0005 |
| *11 Below* | | 0.0009 |
| *11 Above* | | 0.0006 |
| *12 Below* | | 0.0027 |

**Table B-24 Results comparison between models 12B/6/E/T, 12B/6/3101/T and 12B/6/1170/T for Tabas 2 ground acceleration record**

| *Record "Tabas 2"* | | | | |
|---|---|---|---|---|
| | *12B/6/E/T* | | *12B/6/3101/T* | |
| | *Maximum base shear (kN)* | | *Maximum base shear (kN)* | |
| *W/o P-Δ* | 1811.6 | | 1866.5 | |
| *With P-Δ* | 1840.5 | | 1866.0 | |
| *Inter-storey drifts* |  | |  | |
| | *Maximum inter-storey drifts (% of storey height)* | | *Maximum inter-storey drifts (% of storey height)* | |
| *Storey* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* |
| *1* | 0.72 | 0.85 | 0.69 | 0.81 |
| *2* | 0.94 | 1.03 | 0.92 | 1.02 |
| *3* | 0.95 | 0.98 | 0.97 | 0.99 |
| *4* | 1.00 | 0.89 | 1.02 | 0.90 |
| *5* | 0.91 | 0.82 | 0.93 | 0.83 |
| *6* | 0.79 | 0.79 | 0.81 | 0.79 |
| *7* | 0.94 | 1.01 | 0.90 | 1.00 |
| *8* | 1.05 | 1.01 | 1.04 | 1.02 |
| *9* | 0.89 | 0.84 | 0.88 | 0.84 |
| *10* | 0.57 | 0.55 | 0.59 | 0.56 |
| *11* | 0.38 | 0.36 | 0.38 | 0.35 |
| *12* | 0.21 | 0.20 | 0.29 | 0.28 |

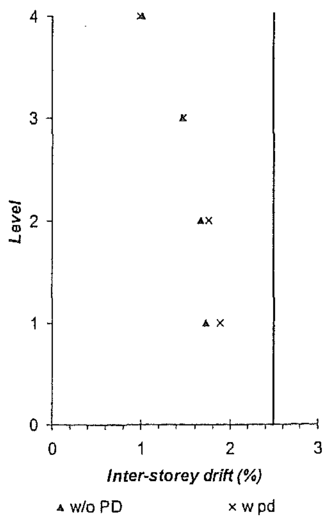
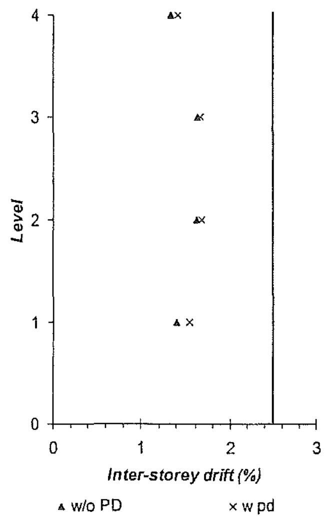
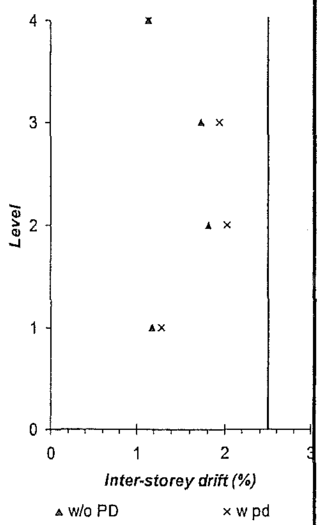
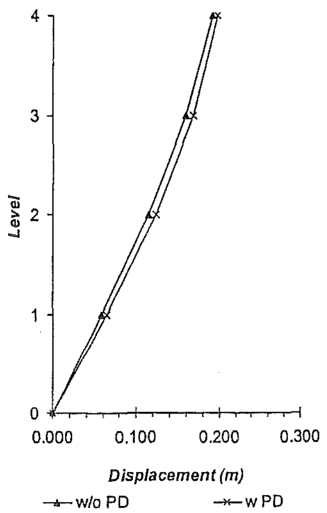
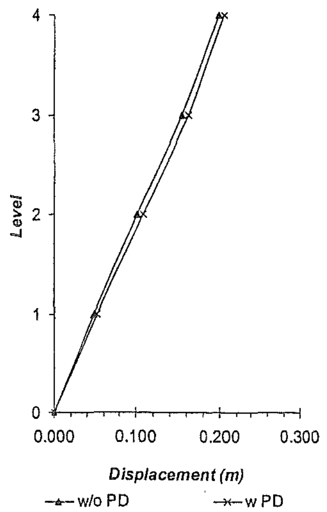
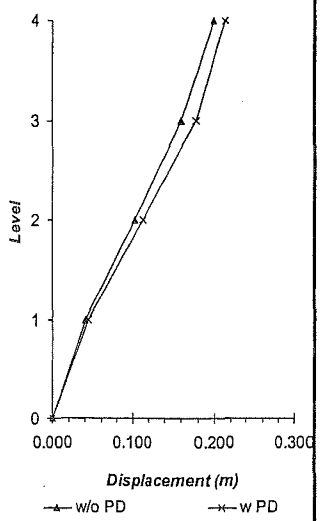
*Record "Tabas 2"*

| | *12B/6/E/T* | | *12B/6/3101/T* | |
|---|---|---|---|---|
| *Maximum displacements* |  | |  | |
| | *Maximum displacements (m)* | | *Maximum displacements (m)* | |
| *Level* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* |
| *1* | 0.025 | 0.029 | 0.023 | 0.027 |
| *2* | 0.056 | 0.063 | 0.054 | 0.061 |
| *3* | 0.086 | 0.095 | 0.086 | 0.094 |
| *4* | 0.114 | 0.120 | 0.116 | 0.120 |
| *5* | 0.144 | 0.139 | 0.146 | 0.139 |
| *6* | 0.166 | 0.150 | 0.168 | 0.151 |
| *7* | 0.183 | 0.169 | 0.185 | 0.169 |
| *8* | 0.199 | 0.199 | 0.200 | 0.199 |
| *9* | 0.208 | 0.226 | 0.209 | 0.226 |
| *10* | 0.215 | 0.243 | 0.215 | 0.242 |
| *11* | 0.224 | 0.251 | 0.223 | 0.250 |
| *12* | 0.229 | 0.255 | 0.230 | 0.256 |
| ***Level*** | ***Centre span beam rotations (rad)*** | | ***Centre span beam rotations (rad)*** | |
| *1* | 0.0084 | | 0.0077 | |
| *2* | 0.0093 | | 0.0093 | |
| *3* | 0.0075 | | 0.0075 | |
| *4* | 0.0073 | | 0.0075 | |
| *5* | 0.0073 | | 0.0072 | |
| *6* | 0.0075 | | 0.0072 | |
| *7* | 0.0111 | | 0.0114 | |
| *8* | 0.0096 | | 0.0102 | |
| *9* | 0.0063 | | 0.0063 | |
| *10* | 0.0031 | | 0.0031 | |
| *11* | 0.0008 | | 0.0009 | |
| *12* | 0.0003 | | 0.0002 | |

| *Record "Tabas 2"* | | |
|---|---|---|
| | *12B/6/E/T* | *12B/6/3101/T* |
| | *Interior column rotation (rad)* | *Interior column rotation (rad)* |
| *G.L.* | 0.0054 | 0.0049 |
| *1 Below* | *Elastic columns at upper levels* | 0.0007 |
| *1 Above* | | 0.0013 |
| *2 Below* | | 0.0011 |
| *2 Above* | | 0.0009 |
| *3 Below* | | 0.0011 |
| *3 Above* | | 0.0010 |
| *4 Below* | | 0.0011 |
| *4 Above* | | 0.0009 |
| *5 Below* | | 0.0010 |
| *5 Above* | | 0.0008 |
| *6 Below* | | 0.0009 |
| *6 Above* | | 0.0014 |
| *7 Below* | | 0.0010 |
| *7 Above* | | 0.0010 |
| *8 Below* | | 0.0010 |
| *8 Above* | | 0.0008 |
| *9 Below* | | 0.0012 |
| *9 Above* | | 0.0009 |
| *10 Below* | | 0.0010 |
| *10 Above* | | 0.0006 |
| *11 Below* | | 0.0009 |
| *11 Above* | | 0.0008 |
| *12 Below* | | 0.0036 |

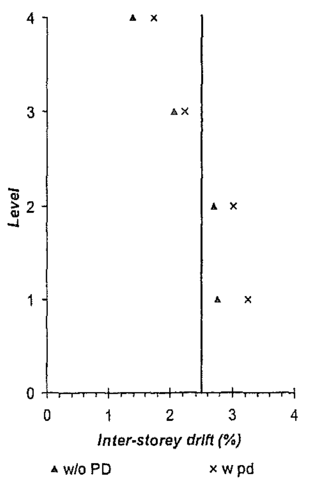
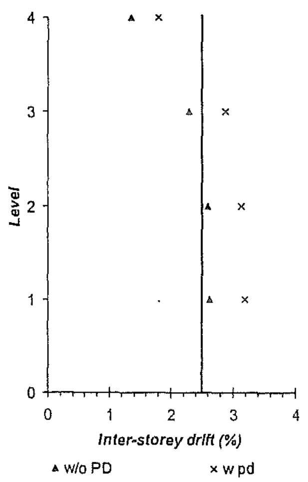
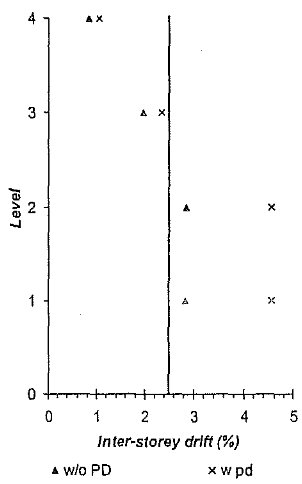
## *B.4 Four storey building*

**Table B-25 Result comparison between models 4/6/E/T, 4/6/3101/T and 4/6/1170/T for Modified El Centro NS ground acceleration record**

| *Record "Modified El Centro NS"* | | | | | | |
|---|---|---|---|---|---|---|
| | *4/6/E/T* | | *4/6/3101/T* | | *4/6/1170/T* | |
| | *Maximum base shear (kN)* | | *Maximum base shear (kN)* | | *Maximum base shear (kN)* | |
| *W/o P-Δ* | 1001.5 | | 959.7 | | 926.7 | |
| *With P-Δ* | 1180.8 | | 1056.6 | | 981.5 | |
| *Inter-storey drifts* |   | |   | |   | |
| | *Maximum inter-storey drifts (% of storey height)* | | *Maximum inter-storey drifts (% of storey height)* | | *Maximum inter-storey drifts (% of storey height)* | |
| *Storey* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* |
| *1* | 1.72 | 1.89 | 1.40 | 1.53 | 1.17 | 1.28 |
| *2* | 1.67 | 1.76 | 1.61 | 1.67 | 1.82 | 2.03 |
| *3* | 1.47 | 1.47 | 1.63 | 1.66 | 1.74 | 1.94 |
| *4* | 1.01 | 0.99 | 1.32 | 1.41 | 1.14 | 1.13 |
| *Maximum displacements* |   | |   | |   | |

| Record "Modified El Centro NS" | | | | | | |
|---|---|---|---|---|---|---|
| | 4/6/E/T | | 4/6/3101/T | | 4/6/1170/T | |
| | Maximum displacements (m) | | Maximum displacements (m) | | Maximum displacements (m) | |
| Level | Without P-Δ | With P-Δ | Without P-Δ | With P-Δ | Without P-Δ | With P-Δ |
| 1 | 0.058 | 0.064 | 0.048 | 0.052 | 0.040 | 0.044 |
| 2 | 0.115 | 0.124 | 0.101 | 0.109 | 0.102 | 0.113 |
| 3 | 0.160 | 0.169 | 0.155 | 0.162 | 0.160 | 0.178 |
| 4 | 0.192 | 0.198 | 0.200 | 0.206 | 0.199 | 0.212 |
| Level | Centre span beam rotations (rad) | | Centre span beam rotations (rad) | | Centre span beam rotations (rad) | |
| 1 | 0.0164 | | 0.0135 | | 0.0109 | |
| 2 | 0.0143 | | 0.0152 | | 0.0181 | |
| 3 | 0.0114 | | 0.0150 | | 0.0119 | |
| 4 | 0.0052 | | 0.0002 | | 0.0002 | |
| | Interior column rotation (rad) | | Interior column rotation (rad) | | Interior column rotation (rad) | |
| G.L. | 0.0139 | | 0.0107 | | 0.0088 | |
| 1 Below | Elastic columns at upper levels | | 0.0010 | | 0.0010 | |
| 1 Above | | | 0.0014 | | 0.0076 | |
| 2 Below | | | 0.0012 | | 0.0022 | |
| 2 Above | | | 0.0010 | | 0.0020 | |
| 3 Below | | | 0.0011 | | 0.0069 | |
| 3 Above | | | 0.0012 | | 0.0029 | |
| 4 Below | | | 0.0116 | | 0.0093 | |

**Table B-26 Result comparison between models 4/6/E/T, 4/6/3101/T and 4/6/1170/T for Modified El Centro EW ground acceleration record**

| Record "Modified El Centro EW" | | | |
|---|---|---|---|
| | 4/6/E/T | 4/6/3101/T | 4/6/1170/T |
| | Maximum base shear (kN) | Maximum base shear (kN) | Maximum base shear (kN) |
| W/o P-Δ | 1201.1 | 1113.0 | 1031.2 |
| With P-Δ | 1512.5 | 1327.3 | 1460.7 |
| Inter-storey drifts |  |  |  |

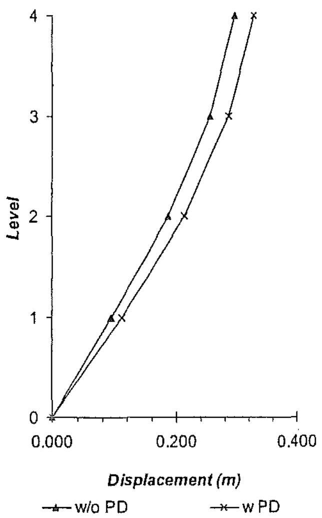
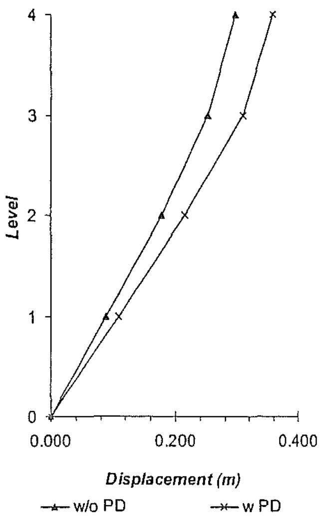
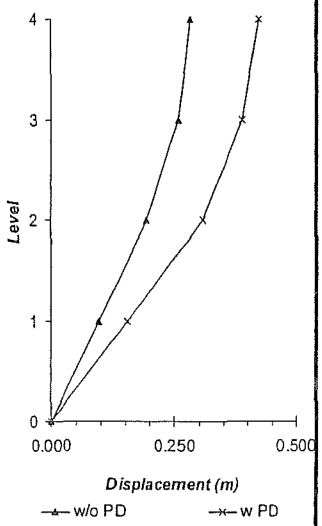
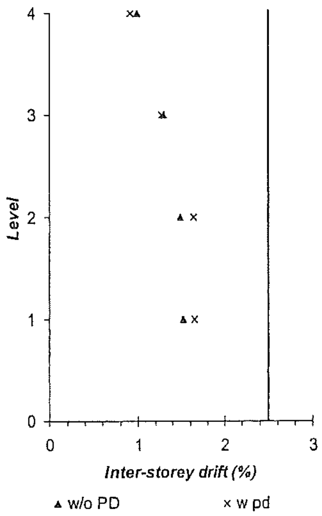
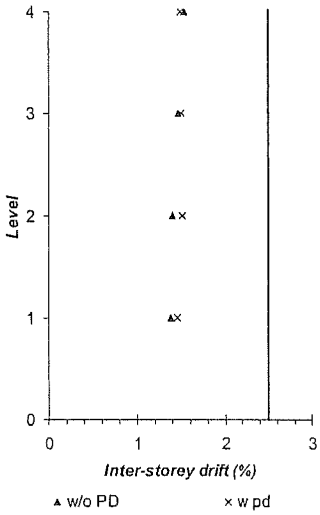
| Record "Modified El Centro EW" | | | | | | |
|---|---|---|---|---|---|---|
| | *4/6/E/T* | | *4/6/3101/T* | | *4/6/1170/T* | |
| | *Maximum inter-storey drifts (% of storey height)* | | *Maximum inter-storey drifts (% of storey height)* | | *Maximum inter-storey drifts (% of storey height)* | |
| *Storey* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* |
| *1* | 2.77 | 3.26 | 2.63 | 3.19 | 2.84 | 4.54 |
| *2* | 2.70 | 3.01 | 2.60 | 3.12 | 2.85 | 4.56 |
| *3* | 2.06 | 2.24 | 2.30 | 2.87 | 1.96 | 2.35 |
| *4* | 1.39 | 1.73 | 1.36 | 1.80 | 0.84 | 1.06 |
| *Maximum displacements* |  | |  | |  | |
| | *Maximum displacements (m)* | | *Maximum displacements (m)* | | *Maximum displacements (m)* | |
| *Level* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* |
| *1* | 0.094 | 0.111 | 0.089 | 0.109 | 0.097 | 0.154 |
| *2* | 0.186 | 0.213 | 0.178 | 0.214 | 0.194 | 0.309 |
| *3* | 0.256 | 0.286 | 0.253 | 0.309 | 0.259 | 0.388 |
| *4* | 0.296 | 0.329 | 0.297 | 0.359 | 0.282 | 0.420 |
| *Level* | *Centre span beam rotations (rad)* | | *Centre span beam rotations (rad)* | | *Centre span beam rotations (rad)* | |
| *1* | 0.0306 | | 0.0303 | | 0.0300 | |
| *2* | 0.0249 | | 0.0294 | | 0.0209 | |
| *3* | 0.0186 | | 0.0179 | | 0.0104 | |
| *4* | 0.0133 | | 0.0002 | | 0.0002 | |
| | *Interior column rotation (rad)* | | *Interior column rotation (rad)* | | *Interior column rotation (rad)* | |
| *G.L.* | 0.0248 | | 0.0243 | | 0.0363 | |
| *1 Below* | | | 0.0014 | | 0.0135 | |
| *1 Above* | | | 0.0014 | | 0.0170 | |
| *2 Below* | | | 0.0016 | | 0.0227 | |
| *2 Above* | *Elastic columns at upper levels* | | 0.0011 | | 0.0033 | |
| *3 Below* | | | 0.0122 | | 0.0118 | |
| *3 Above* | | | 0.0023 | | 0.0010 | |
| *4 Below* | | | 0.0160 | | 0.0093 | |

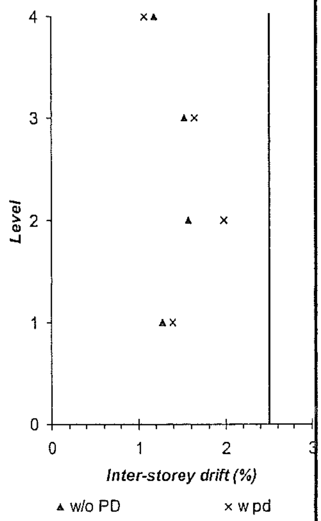
**Table B-27 Result comparison between models 4/6/E/T, 4/6/3101/T and 4/6/1170/T for Modified TAFT 1 ground acceleration record**

| *Record "Modified TAFT1"* | | | |
|---|---|---|---|
| | *4/6/E/T* | *4/6/3101/T* | *4/6/1170/T* |
| | *Maximum base shear (kN)* | *Maximum base shear (kN)* | *Maximum base shear (kN)* |
| *W/o P-Δ* | 1041.6 | 1068.1 | 1004.1 |
| *With P-Δ* | 1183.4 | 1150.5 | 1084.1 |

*Inter-storey drifts*

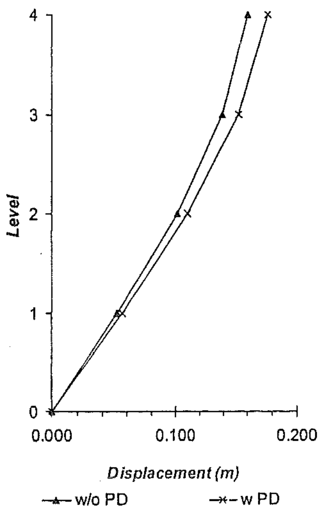


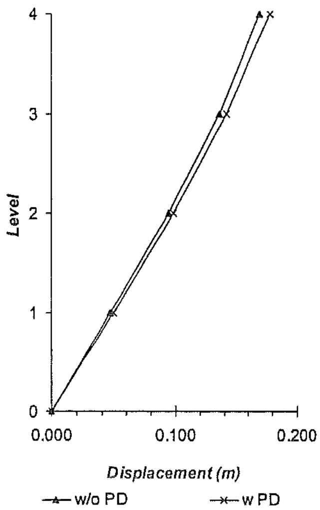



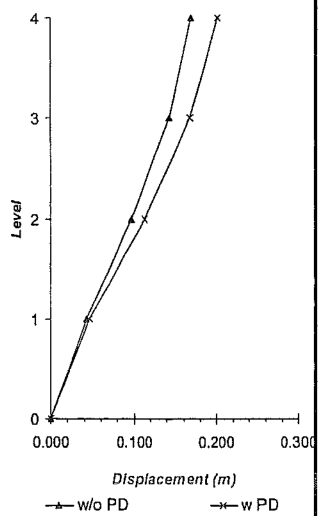




| | *4/6/E/T* | | *4/6/3101/T* | | *4/6/1170/T* | |
|---|---|---|---|---|---|---|
| | *Maximum inter-storey drifts (% of storey height)* | | *Maximum inter-storey drifts (% of storey height)* | | *Maximum inter-storey drifts (% of storey height)* | |
| *Storey* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* |
| *1* | 1.52 | 1.65 | 1.37 | 1.45 | 1.26 | 1.39 |
| *2* | 1.49 | 1.64 | 1.40 | 1.51 | 1.57 | 1.98 |
| *3* | 1.29 | 1.26 | 1.46 | 1.50 | 1.53 | 1.64 |
| *4* | 0.99 | 0.90 | 1.54 | 1.49 | 1.17 | 1.05 |

*Maximum displacements*

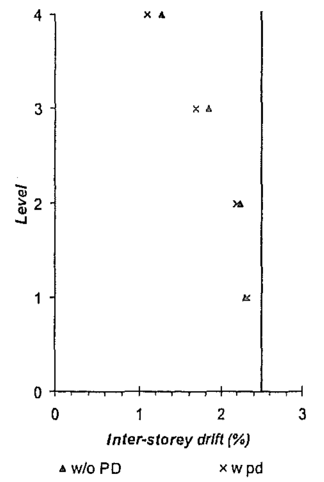
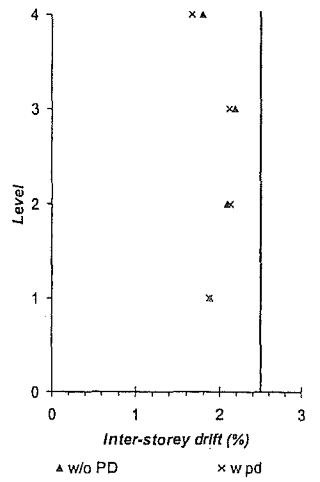
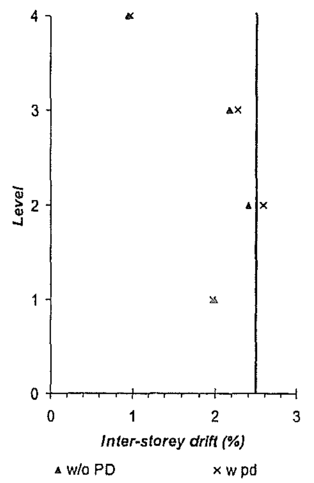


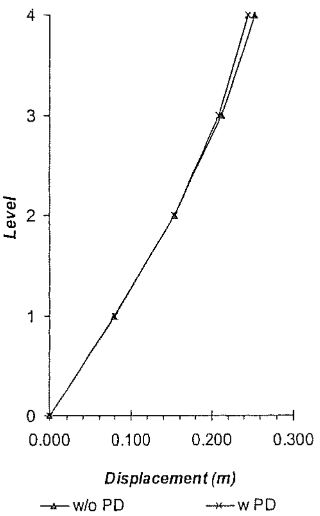
| Record "Modified TAFT1" | | | | | | |
|---|---|---|---|---|---|---|
| | 4/6/E/T | | 4/6/3101/T | | 4/6/1170/T | |
| | Maximum displacements (m) | | Maximum displacements (m) | | Maximum displacements (m) | |
| Level | Without P-Δ | With P-Δ | Without P-Δ | With P-Δ | Without P-Δ | With P-Δ |
| 1 | 0.052 | 0.056 | 0.047 | 0.049 | 0.043 | 0.047 |
| 2 | 0.101 | 0.111 | 0.094 | 0.098 | 0.096 | 0.112 |
| 3 | 0.140 | 0.153 | 0.137 | 0.142 | 0.142 | 0.167 |
| 4 | 0.161 | 0.177 | 0.170 | 0.178 | 0.169 | 0.201 |
| Level | Centre span beam rotations (rad) | | Centre span beam rotations (rad) | | Centre span beam rotations (rad) | |
| 1 | 0.0157 | | 0.0122 | | 0.0113 | |
| 2 | 0.0135 | | 0.0133 | | 0.0149 | |
| 3 | 0.0105 | | 0.0146 | | 0.0096 | |
| 4 | 0.0088 | | 0.0002 | | 0.0002 | |
| | Interior column rotation (rad) | | Interior column rotation (rad) | | Interior column rotation (rad) | |
| G.L. | 0.0123 | | 0.0110 | | 0.0096 | |
| 1 Below | Elastic columns at upper levels | | 0.0012 | | 0.0015 | |
| 1 Above | | | 0.0014 | | 0.0065 | |
| 2 Below | | | 0.0013 | | 0.0047 | |
| 2 Above | | | 0.0013 | | 0.0017 | |
| 3 Below | | | 0.0016 | | 0.0069 | |
| 3 Above | | | 0.0013 | | 0.0023 | |
| 4 Below | | | 0.0131 | | 0.0082 | |

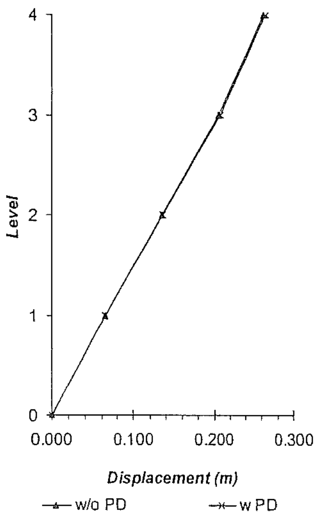
**Table B-28 Result comparison between models 4/6/E/T, 4/6/3101/T and 4/6/1170/T for Modified TAFT 2 ground acceleration record**

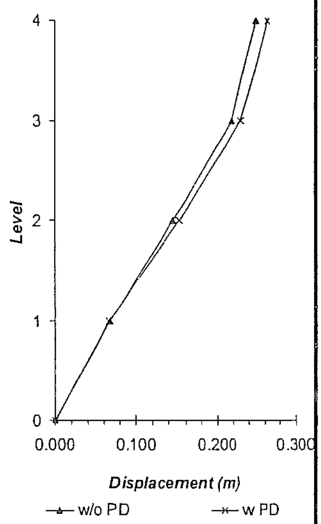
| Record "Modified TAFT2" | | | |
|---|---|---|---|
| | 4/6/E/T | 4/6/3101/T | 4/6/1170/T |
| | Maximum base shear (kN) | Maximum base shear (kN) | Maximum base shear (kN) |
| W/o P-Δ | 1026.5 | 1050.4 | 953.7 |
| With P-Δ | 1242.3 | 1099.5 | 1085.8 |
| Inter-storey drifts |   |   |   |

| Record "Modified TAFT2" | | | | | | |
|---|---|---|---|---|---|---|
| | *4/6/E/T* | | *4/6/3101/T* | | *4/6/1170/T* | |
| | *Maximum inter-storey drifts (% of storey height)* | | *Maximum inter-storey drifts (% of storey height)* | | *Maximum inter-storey drifts (% of storey height)* | |
| *Storey* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* |
| *1* | 2.29 | 2.32 | 1.89 | 1.87 | 1.99 | 1.97 |
| *2* | 2.24 | 2.19 | 2.09 | 2.12 | 2.41 | 2.58 |
| *3* | 1.84 | 1.67 | 2.18 | 2.11 | 2.16 | 2.26 |
| *4* | 1.26 | 1.09 | 1.81 | 1.66 | 0.94 | 0.96 |

*Maximum displacements*



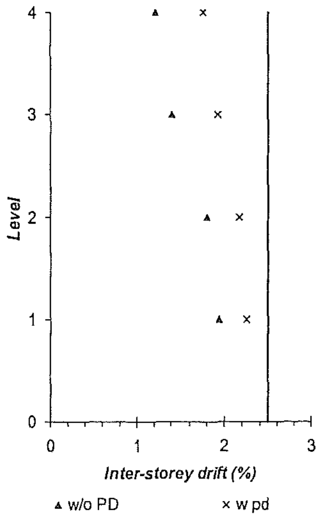

| | *4/6/E/T* | | *4/6/3101/T* | | *4/6/1170/T* | |
|---|---|---|---|---|---|---|
| | *Maximum displacements (m)* | | *Maximum displacements (m)* | | *Maximum displacements (m)* | |
| *Level* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* |
| *1* | 0.078 | 0.079 | 0.064 | 0.064 | 0.068 | 0.067 |
| *2* | 0.154 | 0.154 | 0.135 | 0.136 | 0.145 | 0.153 |
| *3* | 0.214 | 0.209 | 0.205 | 0.208 | 0.218 | 0.230 |
| *4* | 0.253 | 0.244 | 0.263 | 0.264 | 0.248 | 0.262 |
| *Level* | *Centre span beam rotations (rad)* | | *Centre span beam rotations (rad)* | | *Centre span beam rotations (rad)* | |
| *1* | 0.0212 | | 0.0177 | | 0.0181 | |
| *2* | 0.0179 | | 0.0200 | | 0.0211 | |
| *3* | 0.0129 | | 0.0179 | | 0.0092 | |
| *4* | 0.0061 | | 0.0002 | | 0.0002 | |
| | *Interior column rotation (rad)* | | *Interior column rotation (rad)* | | *Interior column rotation (rad)* | |
| *G.L.* | 0.0180 | | 0.0138 | | 0.0149 | |
| *1 Below* | *Elastic columns at upper levels* | | 0.0011 | | 0.0010 | |
| *1 Above* | | | 0.0014 | | 0.0073 | |
| *2 Below* | | | 0.0014 | | 0.0041 | |
| *2 Above* | | | 0.0011 | | 0.0020 | |
| *3 Below* | | | 0.0038 | | 0.0123 | |
| *3 Above* | | | 0.0009 | | 0.0007 | |
| *4 Below* | | | 0.0147 | | 0.0080 | |

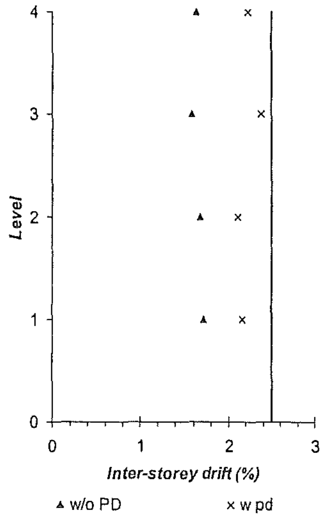
**Table B-29 Result comparison between models 4/6/E/T, 4/6/3101/T and 4/6/1170/T for El Centro 1 ground acceleration record**

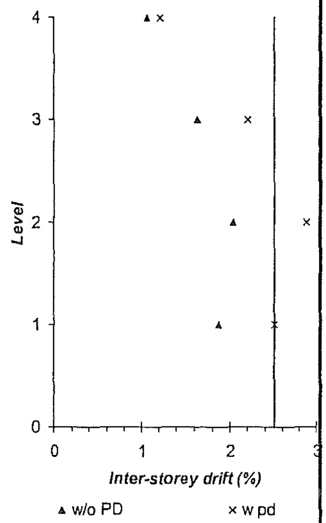
*Record "El Centro 1"*

| | *4/6/E/T* | *4/6/3101/T* | *4/6/1170/T* |
|---|---|---|---|
| | *Maximum base shear (kN)* | *Maximum base shear (kN)* | *Maximum base shear (kN)* |
| *W/o P-Δ* | 1095.6 | 1058.1 | 999.3 |
| *With P-Δ* | 1312.4 | 1251.5 | 1200.5 |

*Inter-storey drifts*

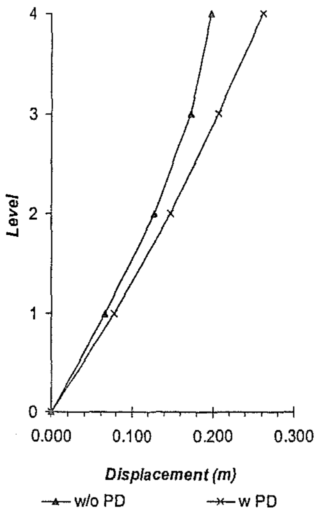


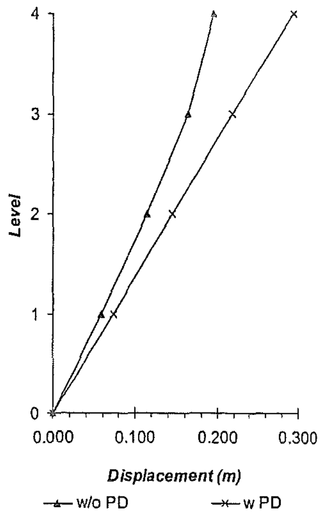



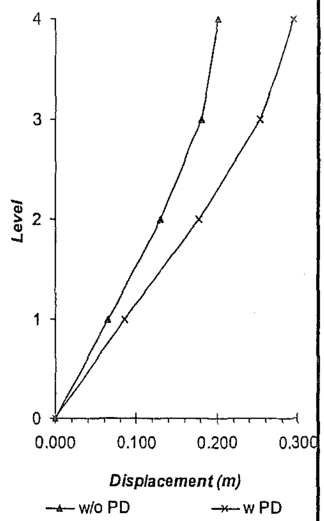




| | *4/6/E/T* | | *4/6/3101/T* | | *4/6/1170/T* | |
|---|---|---|---|---|---|---|
| | *Maximum inter-storey drifts (% of storey height)* | | *Maximum inter-storey drifts (% of storey height)* | | *Maximum inter-storey drifts (% of storey height)* | |
| *Storey* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* |
| *1* | 1.94 | 2.25 | 1.71 | 2.16 | 1.88 | 2.50 |
| *2* | 1.80 | 2.17 | 1.67 | 2.11 | 2.04 | 2.88 |
| *3* | 1.40 | 1.94 | 1.59 | 2.37 | 1.63 | 2.21 |
| *4* | 1.21 | 1.76 | 1.63 | 2.22 | 1.06 | 1.21 |

*Maximum displacements*



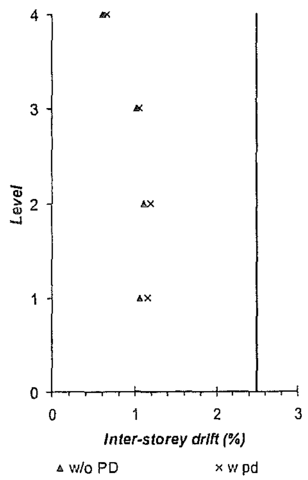
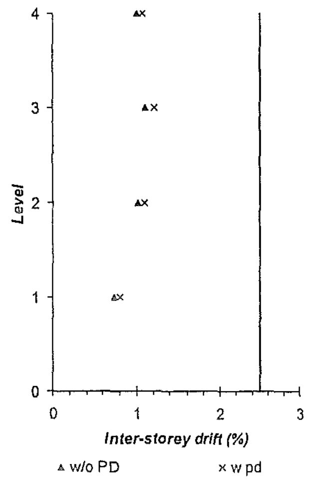
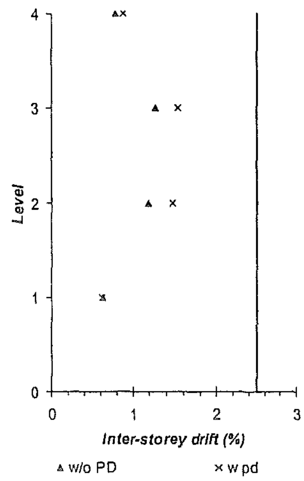




*Record "El Centro 1"*

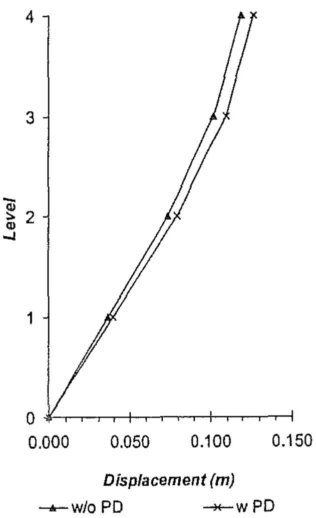
| | *4/6/E/T* | | *4/6/3101/T* | | *4/6/1170/T* | |
|---|---|---|---|---|---|---|
| | *Maximum displacements (m)* | | *Maximum displacements (m)* | | *Maximum displacements (m)* | |
| *Level* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* |
| *1* | 0.0660 | 0.076 | 0.058 | 0.073 | 0.064 | 0.085 |
| *2* | 0.1273 | 0.147 | 0.115 | 0.144 | 0.130 | 0.177 |
| *3* | 0.1728 | 0.207 | 0.163 | 0.218 | 0.180 | 0.252 |
| *4* | 0.1973 | 0.261 | 0.196 | 0.293 | 0.200 | 0.293 |
| *Level* | *Centre span beam rotations (rad)* | | *Centre span beam rotations (rad)* | | *Centre span beam rotations (rad)* | |
| *1* | 0.0203 | | 0.0194 | | 0.0213 | |
| *2* | 0.0184 | | 0.0202 | | 0.0178 | |
| *3* | 0.0182 | | 0.0211 | | 0.0117 | |
| *4* | 0.0133 | | 0.0002 | | 0.0002 | |
| | *Interior column rotation (rad)* | | *Interior column rotation (rad)* | | *Interior column rotation (rad)* | |
| *G.L.* | 0.0170 | | 0.0162 | | 0.0189 | |
| *1 Below* | *Elastic columns at upper levels* | | 0.0013 | | 0.0032 | |
| *1 Above* | | | 0.0012 | | 0.0080 | |
| *2 Below* | | | 0.0015 | | 0.0102 | |
| *2 Above* | | | 0.0014 | | 0.0035 | |
| *3 Below* | | | 0.0043 | | 0.0103 | |
| *3 Above* | | | 0.0024 | | 0.0008 | |
| *4 Below* | | | 0.0201 | | 0.0104 | |

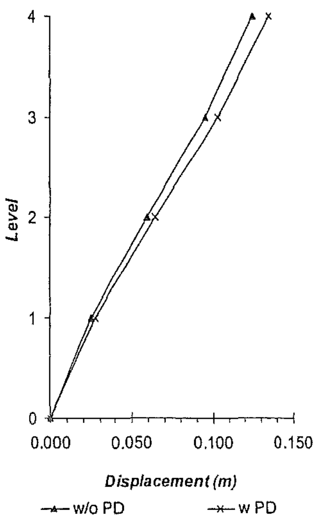
**Table B-30 Result comparison between models 4/6/E/T, 4/6/3101/T and 4/6/1170/T for El Centro 2 ground acceleration record**

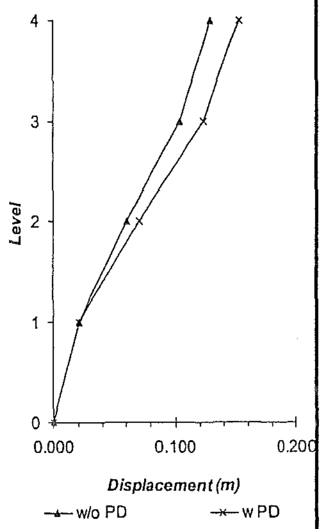
| *Record "El Centro 2"* | | | |
|---|---|---|---|
| | *4/6/E/T* | *4/6/3101/T* | *4/6/1170/T* |
| | *Maximum base shear (kN)* | *Maximum base shear (kN)* | *Maximum base shear (kN)* |
| *W/o P-Δ* | 917.3 | 937.0 | 923.5 |
| *With P-Δ* | 1021.0 | 984.5 | 956.2 |
| *Inter-storey drifts* |  |  |  |

| Record "El Centro 2" | | | | | | |
|---|---|---|---|---|---|---|
| | *4/6/E/T* | | *4/6/3101/T* | | *4/6/1170/T* | |
| | *Maximum inter-storey drifts (% of storey height)* | | *Maximum inter-storey drifts (% of storey height)* | | *Maximum inter-storey drifts (% of storey height)* | |
| *Storey* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* |
| *1* | 1.06 | 1.15 | 0.73 | 0.79 | 0.63 | 0.61 |
| *2* | 1.11 | 1.18 | 1.01 | 1.09 | 1.19 | 1.47 |
| *3* | 1.02 | 1.06 | 1.11 | 1.21 | 1.27 | 1.54 |
| *4* | 0.60 | 0.64 | 1.00 | 1.07 | 0.77 | 0.87 |

*Maximum displacements*



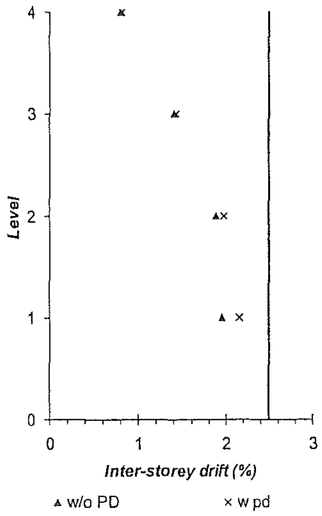

| | *4/6/E/T* | | *4/6/3101/T* | | *4/6/1170/T* | |
|---|---|---|---|---|---|---|
| | *Maximum displacements (m)* | | *Maximum displacements (m)* | | *Maximum displacements (m)* | |
| *Level* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* |
| *1* | 0.036 | 0.039 | 0.025 | 0.027 | 0.021 | 0.021 |
| *2* | 0.074 | 0.079 | 0.059 | 0.064 | 0.061 | 0.071 |
| *3* | 0.102 | 0.110 | 0.095 | 0.104 | 0.103 | 0.123 |
| *4* | 0.119 | 0.126 | 0.125 | 0.135 | 0.129 | 0.152 |
| Level | Centre span beam rotations *(rad)* | | Centre span beam rotations *(rad)* | | Centre span beam rotations *(rad)* | |
| *1* | 0.0091 | | 0.0066 | | 0.0042 | |
| *2* | 0.0095 | | 0.0094 | | 0.0132 | |
| *3* | 0.0074 | | 0.0103 | | 0.0094 | |
| *4* | 0.0019 | | 0.0002 | | 0.0002 | |
| | *Interior column rotation (rad)* | | *Interior column rotation (rad)* | | *Interior column rotation (rad)* | |
| *G.L.* | 0.0077 | | 0.0046 | | 0.0034 | |
| *1 Below* | *Elastic columns at upper levels* | | 0.0010 | | 0.0010 | |
| *1 Above* | | | 0.0014 | | 0.0073 | |
| *2 Below* | | | 0.0012 | | 0.0014 | |
| *2 Above* | | | 0.0010 | | 0.0016 | |
| *3 Below* | | | 0.0010 | | 0.0051 | |
| *3 Above* | | | 0.0009 | | 0.0019 | |
| *4 Below* | | | 0.0093 | | 0.0065 | |

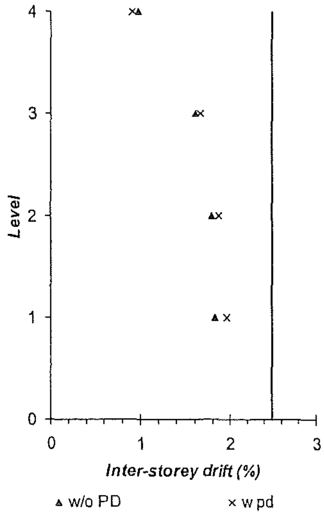
**Table B-31 Result comparison between models 4/6/E/T, 4/6/3101/T and 4/6/1170/T for Tabas 1 ground acceleration record**

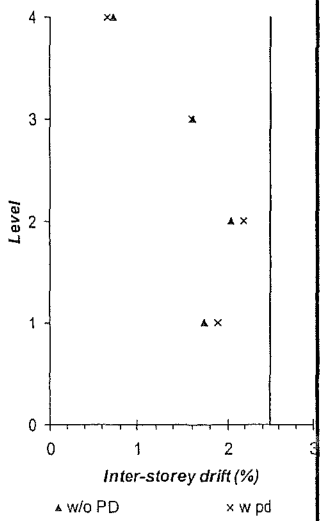
*Record "Tabas 1"*

| | *4/6/E/T* | *4/6/3101/T* | *4/6/1170/T* |
|---|---|---|---|
| | *Maximum base shear (kN)* | *Maximum base shear (kN)* | *Maximum base shear (kN)* |
| *W/o P-Δ* | 1152.6 | 1045.4 | 960.0 |
| *With P-Δ* | 1350.0 | 1184.1 | 1079.3 |

*Inter-storey drifts*

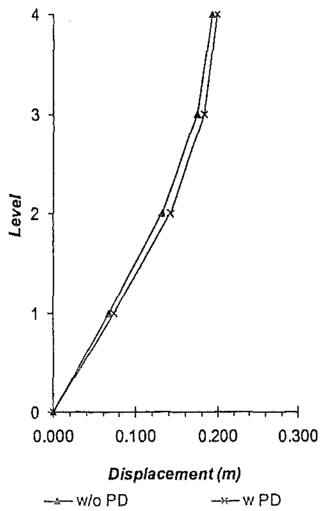


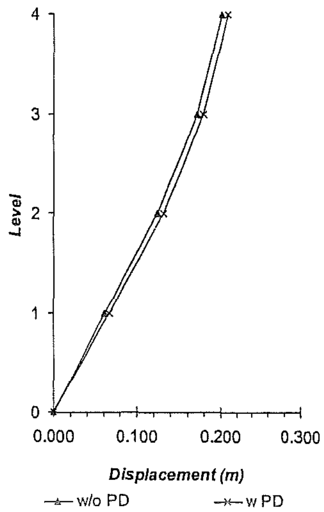



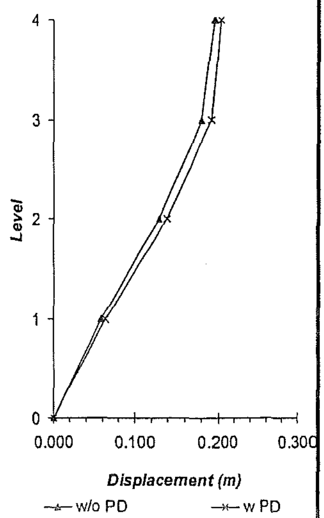




| | *Maximum inter-storey drifts (% of storey height)* | | *Maximum inter-storey drifts (% of storey height)* | | *Maximum inter-storey drifts (% of storey height)* | |
|---|---|---|---|---|---|---|
| *Storey* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* |
| *1* | 1.96 | 2.16 | 1.82 | 1.97 | 1.74 | 1.89 |
| *2* | 1.89 | 1.98 | 1.80 | 1.88 | 2.04 | 2.19 |
| *3* | 1.39 | 1.42 | 1.60 | 1.66 | 1.60 | 1.60 |
| *4* | 0.80 | 0.81 | 0.97 | 0.91 | 0.73 | 0.65 |

*Maximum displacements*

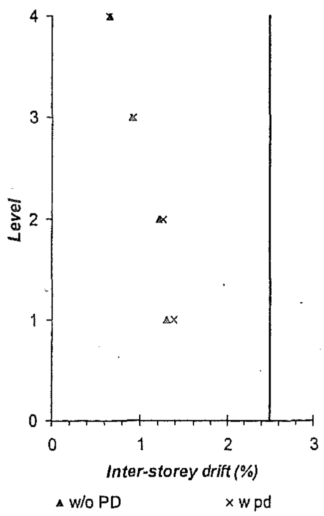
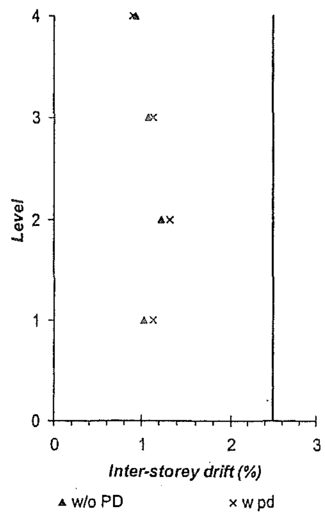
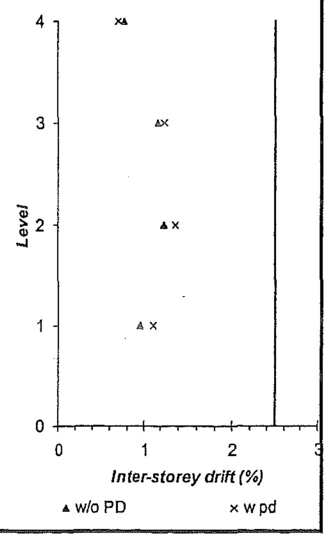


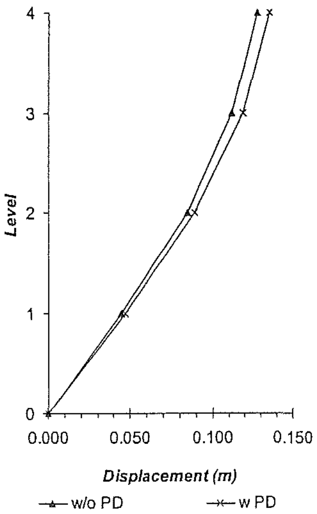
| Record "Tabas 1" | | | | | | |
|---|---|---|---|---|---|---|
| | *4/6/E/T* | | *4/6/3101/T* | | *4/6/1170/T* | |
| | *Maximum displacements (m)* | | *Maximum displacements (m)* | | *Maximum displacements (m)* | |
| *Level* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* |
| *1* | 0.067 | 0.073 | 0.062 | 0.067 | 0.059 | 0.064 |
| *2* | 0.131 | 0.141 | 0.123 | 0.131 | 0.129 | 0.139 |
| *3* | 0.175 | 0.183 | 0.171 | 0.179 | 0.181 | 0.191 |
| *4* | 0.193 | 0.199 | 0.202 | 0.209 | 0.195 | 0.203 |
| *Level* | *Centre span beam rotations (rad)* | | *Centre span beam rotations (rad)* | | *Centre span beam rotations (rad)* | |
| *1* | 0.0192 | | 0.0176 | | 0.0159 | |
| *2* | 0.0149 | | 0.0154 | | 0.0156 | |
| *3* | 0.0096 | | 0.0094 | | 0.0049 | |
| *4* | 0.0043 | | 0.0002 | | 0.0002 | |
| | *Interior column rotation (rad)* | | *Interior column rotation (rad)* | | *Interior column rotation (rad)* | |
| *G.L.* | 0.0158 | | 0.0143 | | 0.0137 | |
| *1 Below* | *Elastic columns at upper levels* | | 0.0012 | | 0.0016 | |
| *1 Above* | | | 0.0015 | | 0.0056 | |
| *2 Below* | | | 0.0015 | | 0.0054 | |
| *2 Above* | | | 0.0013 | | 0.0020 | |
| *3 Below* | | | 0.0054 | | 0.0105 | |
| *3 Above* | | | 0.0009 | | 0.0011 | |
| *4 Below* | | | 0.0078 | | 0.0055 | |

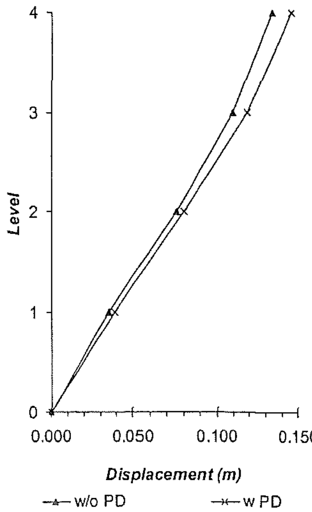
**Table B-32 Result comparison between models 4/6/E/T, 4/6/3101/T and 4/6/1170/T for Tabas 2 ground acceleration record**

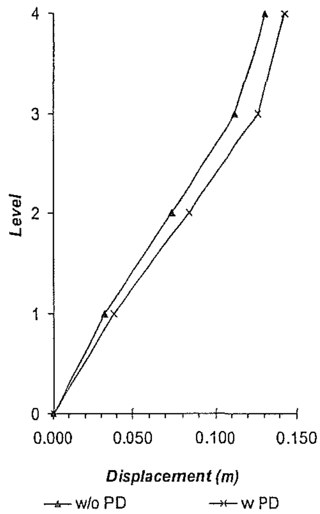
| Record "Tabas 2" | | | |
|---|---|---|---|
| | *4/6/E/T* | *4/6/3101/T* | *4/6/1170/T* |
| | *Maximum base shear (kN)* | *Maximum base shear (kN)* | *Maximum base shear (kN)* |
| *W/o P-Δ* | 1077.1 | 950.7 | 930.1 |
| *With P-Δ* | 1145.2 | 1046.2 | 1012.5 |
| *Inter-storey drifts* |   |   |   |

| Record "Tabas 2" | | | | | | |
|---|---|---|---|---|---|---|
| | *4/6/E/T* | | *4/6/3101/T* | | *4/6/1170/T* | |
| | *Maximum inter-storey drifts (% of storey height)* | | *Maximum inter-storey drifts (% of storey height)* | | *Maximum inter-storey drifts (% of storey height)* | |
| *Storey* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* |
| *1* | 1.29 | 1.37 | 1.02 | 1.13 | 0.94 | 1.10 |
| *2* | 1.20 | 1.26 | 1.22 | 1.31 | 1.23 | 1.37 |
| *3* | 0.90 | 0.91 | 1.06 | 1.13 | 1.16 | 1.23 |
| *4* | 0.66 | 0.64 | 0.93 | 0.89 | 0.76 | 0.70 |

*Maximum displacements*




| | *4/6/E/T* | | *4/6/3101/T* | | *4/6/1170/T* | |
|---|---|---|---|---|---|---|
| | *Maximum displacements (m)* | | *Maximum displacements (m)* | | *Maximum displacements (m)* | |
| *Level* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* | *Without P-Δ* | *With P-Δ* |
| *1* | 0.044 | 0.047 | 0.035 | 0.038 | 0.032 | 0.038 |
| *2* | 0.085 | 0.089 | 0.075 | 0.080 | 0.074 | 0.084 |
| *3* | 0.112 | 0.119 | 0.110 | 0.119 | 0.112 | 0.126 |
| *4* | 0.127 | 0.135 | 0.134 | 0.145 | 0.130 | 0.142 |

| *Level* | *Centre span beam rotations (rad)* | *Centre span beam rotations (rad)* | *Centre span beam rotations (rad)* |
|---|---|---|---|
| *1* | 0.0110 | 0.0090 | 0.0088 |
| *2* | 0.0086 | 0.0103 | 0.0103 |
| *3* | 0.0060 | 0.0075 | 0.0056 |
| *4* | 0.0021 | 0.0002 | 0.0002 |

| | *Interior column rotation (rad)* | *Interior column rotation (rad)* | *Interior column rotation (rad)* |
|---|---|---|---|
| *G.L.* | 0.0098 | 0.0075 | 0.0074 |
| *1 Below* | *Elastic columns at upper levels* | 0.0011 | 0.0010 |
| *1 Above* | | 0.0014 | 0.0029 |
| *2 Below* | | 0.0012 | 0.0022 |
| *2 Above* | | 0.0014 | 0.0026 |
| *3 Below* | | 0.0022 | 0.0051 |
| *3 Above* | | 0.0014 | 0.0013 |
| *4 Below* | | 0.0078 | 0.0059 |

## APPENDIX C. DAMPING FORCES

Table C.1 shows the maximum base shear and the maximum of the sum of the damping forces at all nodes of structures 6/6/3101/T, 12S/6/3101/T, 12B/6/3101/T and 4/6/3101/T respectively, for the non-linear time-history analyses using the modified ground acceleration record El Centro EW with P-delta effects. The damping for all structures was modelled as a constant 5% damping in all the vibration modes. The maximum damping forces are of the order of 70% of the maximum base shear for structures 6/6/3101/T and 4/6/3101/T and around 95% for structures 12A/6/3101/T and 12B/6/3101/T.

**Table C-1 Maximum base shear and maximum damping forces from non-linear time-history analyses when using the modified ground acceleration record El Centro EW**

| *Structure* | *Maximum Base Shear (kN)* | *Damping forces (kN)* |
|---|---|---|
| | | |
| ***6/6/3101/T*** | 1910 | 1421 |
| ***12A/6/3101/T*** | 2430 | 2227 |
| ***12B/6/3101/T*** | 2110 | 2080 |
| ***4/6/3101/T*** | 1330 | 942 |

Figures C.1 to C.5 show on the first graph of each figure, the base shear and sum of the damping forces at each node for the non-linear time-history analyses using the modified ground acceleration record El Centro EW with P-delta effects for structures 6/6/3101/T, 12S/6/3101/T, 12B/6/3101/T and 4/6/3101/T respectively. It can be seen that at the time when a peak on the base shear appears, the damping forces tend to zero. This effect occurs because damping was modelled as viscous and hence it is velocity dependant. The second graph shows the base shear and the subtraction of the base shear and damping forces. To get clearer graphics, only the first ten seconds of each graph are plotted but the entire records were run. The maximum values when the damping forces are subtracted from the base shears are 3088 kN, 4388 kN, 4003 kN and 2218 kN for structures 6/6/3101/T, 12S/6/3101/T, 12B/6/3101/T and 4/6/3101/T respectively. These values show how important the damping forces are to the analyses and the great effect they have to the base shear.

At the time there still exists the uncertainty of how damping works in structures subject to dynamic effects and of the nature of its source. Further studies are needed in this area to verify the damping in structures and get a more accurate way to represent it in analytical models.

6/6/3101/T

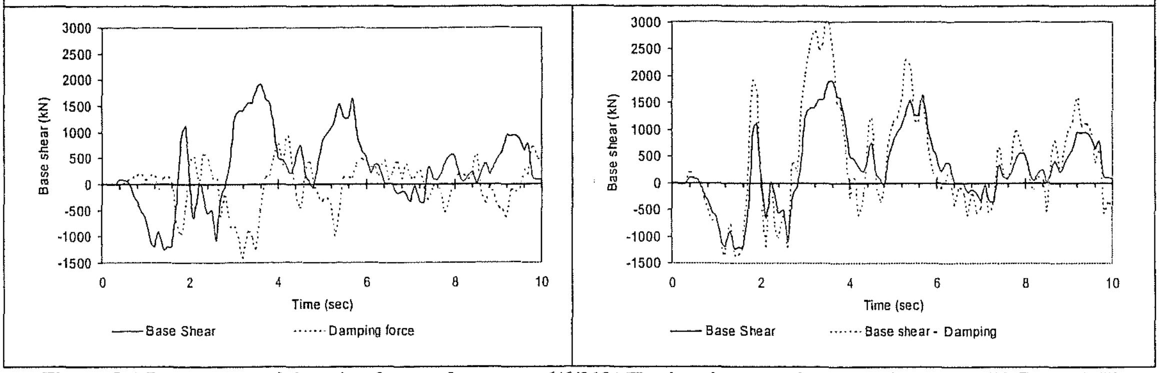



**Figure C-1 Base shear and damping forces of structure 6/6/3101/T using the ground acceleration record El Centro EW**

12A/6/3101/T

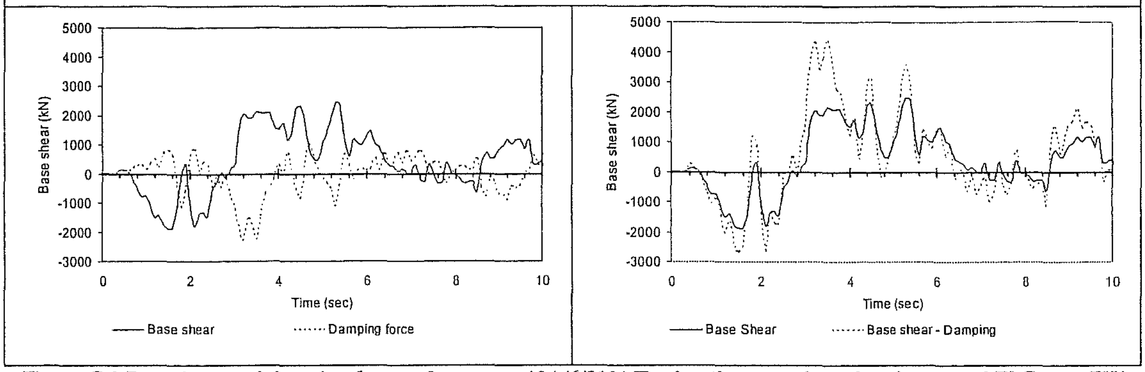



**Figure C-2 Base shear and damping forces of structure 12A/6/3101/T using the ground acceleration record El Centro EW**

12B/6/3101/T

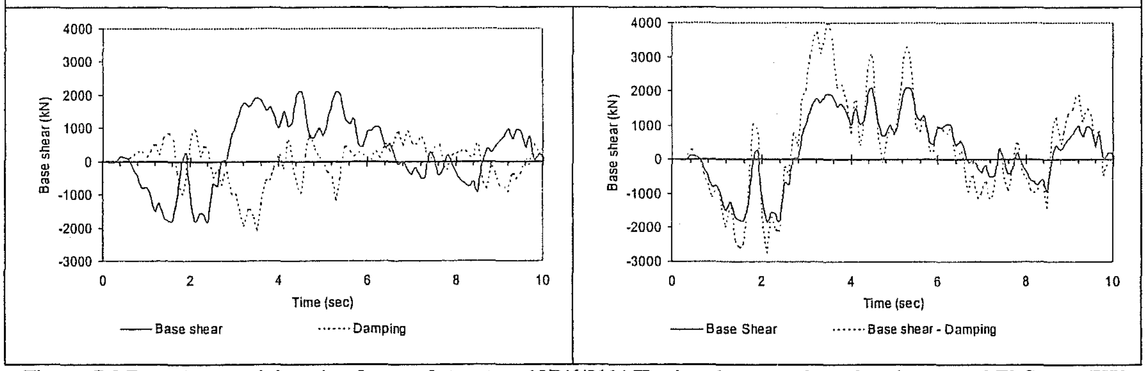



**Figure C-3 Base shear and damping forces of structure 12B/6/3101/T using the ground acceleration record El Centro EW**

4/6/3101/T

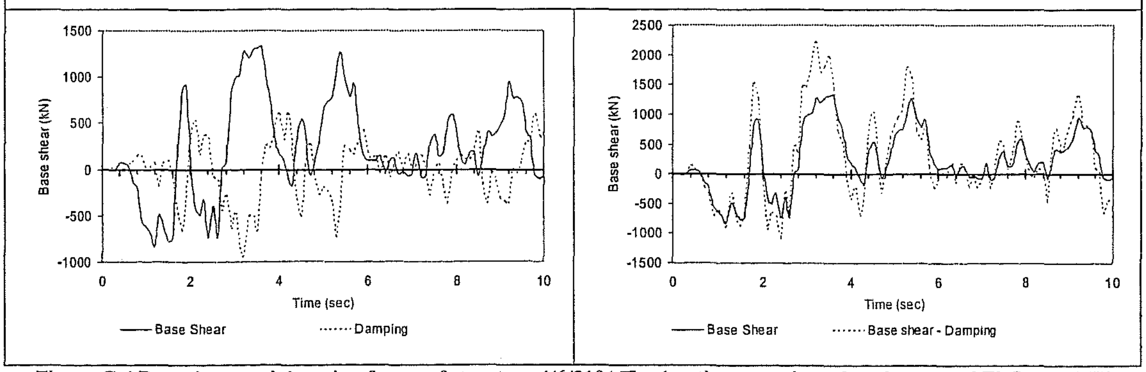



**Figure C-4 Base shear and damping forces of structure 4/6/3101/T using the ground acceleration record El Centro EW**

# APPENDIX D. DISPLACEMENT RESPONSE SPECTRA

Figure D.1 and D.2 show the elastic displacement response spectra for the modified and the natural ground acceleration records respectively used in this work. These spectra correspond to a 5% damping and are scaled to a zone factor of 1.2 from the NZS 4203:1992 (Standards New Zealand 1992). The natural records are scaled according to the scale factor k1 defined for the 6 storey building.

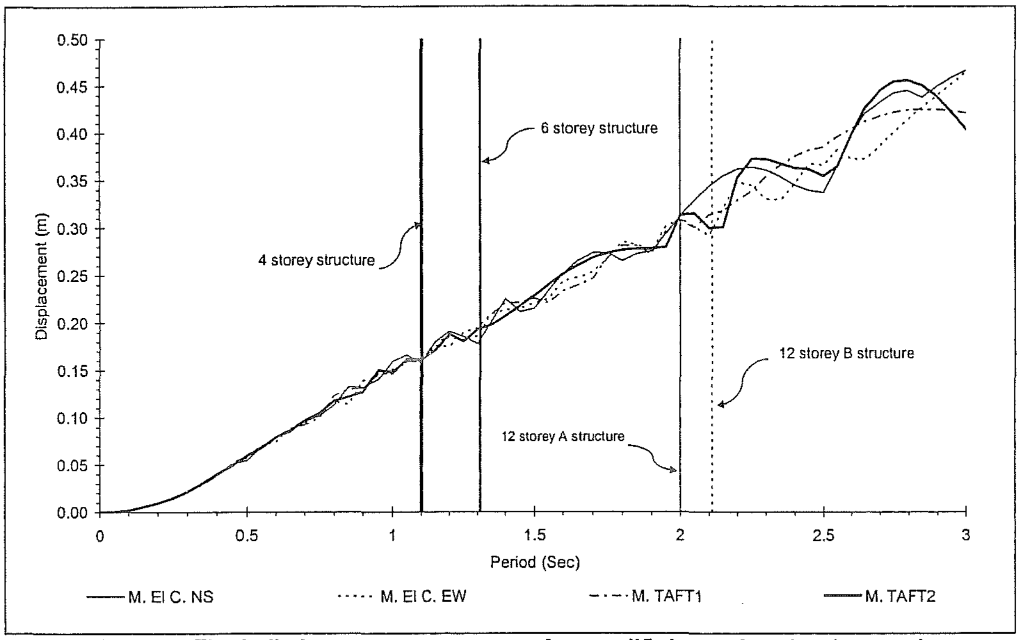



**Figure D-1 Elastic displacement response spectra from modified ground acceleration records**

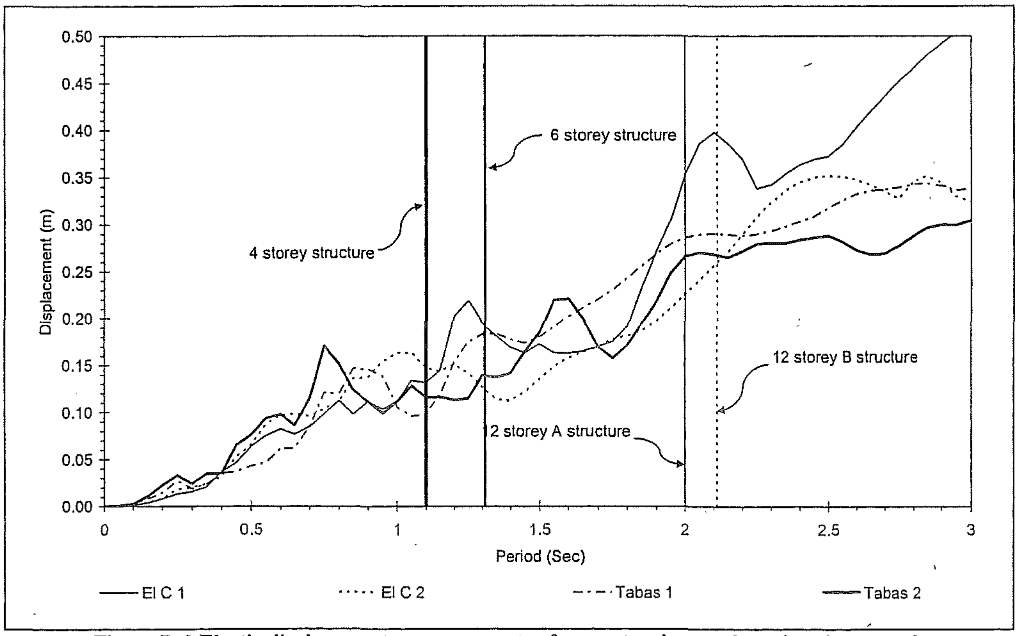



**Figure D-2 Elastic displacement response spectra from natural ground acceleration records**

From these figures, it can be seen that as expected, for elastic single degree of freedom structures, no significant scatter in the displacements will be obtained if the modified ground acceleration records are used. From the way the records are modified, the displacement response spectra keep increasing even for high periods where the displacement of the structure would be equal to the maximum ground displacement. The displacement response spectra from the natural ground acceleration records give an insight on why the displacements from the non-linear time-history analyses when using these records were scattered. As the period increases, the scatter seems to be greater. Figure D-3 shows the displacement response spectra of figures D.1 and D.2 plotted together. This figure illustrates how after a period of 1 second, the modified ground acceleration records are an upper bound limit for the natural records. The natural record El Centro 1 is an exception and for periods around 2 and 2.2 (such as those from the 12 storey A and B structures analysed in this research work) its ordinates are greater than those for the modified ground acceleration records. These results agree with those from the non-linear time-history analyses performed to the different structures throughout this work structures.

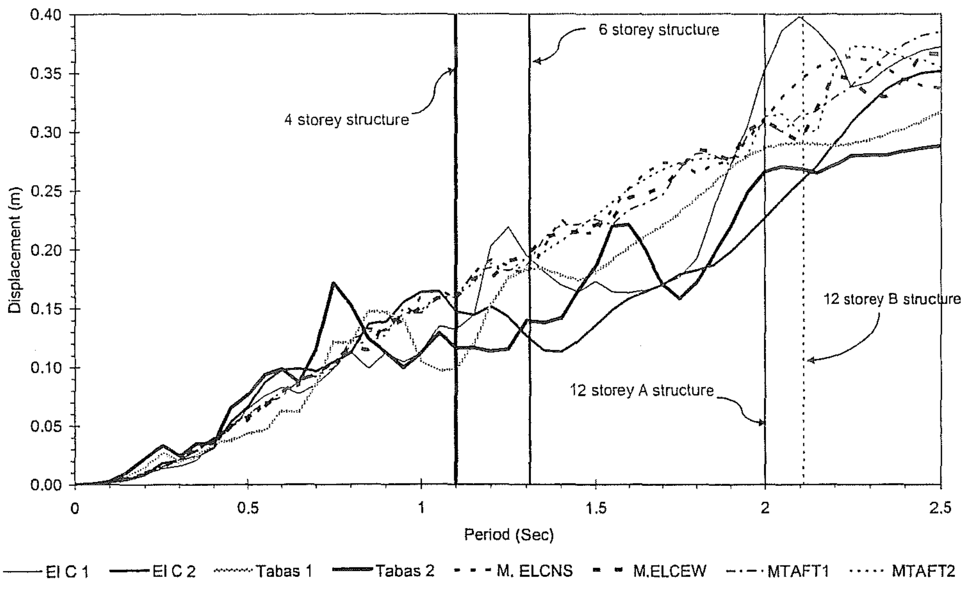



**Figure D-3 Elastic displacement response spectra for the eight ground acceleration records**

To get a better insight of why such a great scatter in the results of the time-history non-linear analysis was obtained when using the modified ground acceleration records, the inelastic displacement and acceleration response spectra for these records was obtained. Figure D.4 shows the response spectra from NZS 4203:1992 (Standards New Zealand 1992) for intermediate soils with a zone factor of 1 and the corresponding reduced response spectra for a ductility of 6. It also shows the inelastic acceleration response spectra for the modified ground acceleration records for a ductility of 6 and a Takeda hysteretic rule as defined for the columns of the structures analysed in the main body of this work. All the acceleration response spectra coming from the modified ground acceleration records are above the inelastic design response spectra defined by the Loadings Standard for a ductility of 6

(Standards New Zealand 1992). The response when these records are used would be overestimated for structures with short periods. Figure D.5 shows that even for single degree of freedom structures the maximum displacements obtained when using the modified ground acceleration records would have a large scatter. The tendency sought throughout this work that the modified El Centro EW ground acceleration record gives the most critical response is confirmed in this figure. The three other records seem to have a small scatter in the periods close to those from the 4 storey and 6 storey structures but a greater scatter in the periods close to the two twelve storey structures.

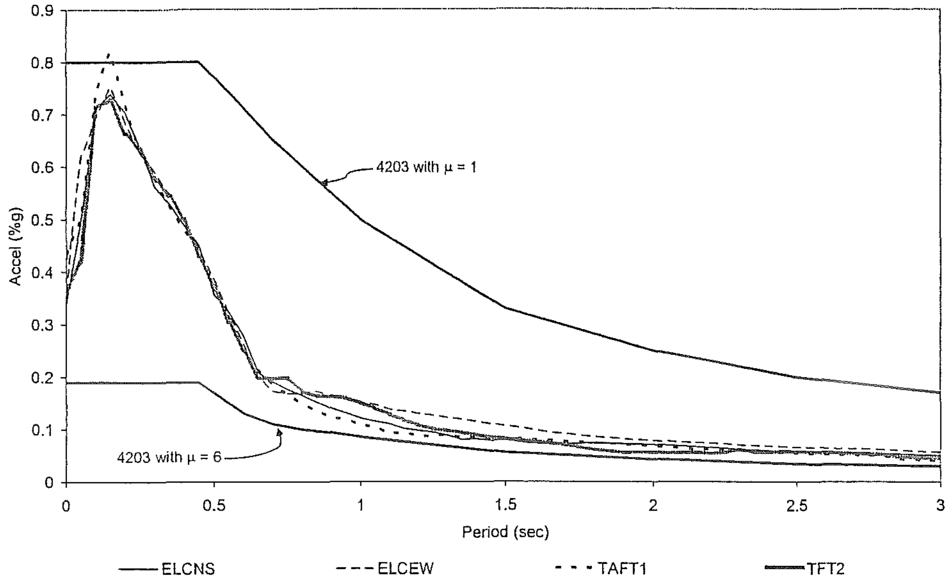



**Figure D-4 Inelastic acceleration response spectra for modified ground acceleration records**

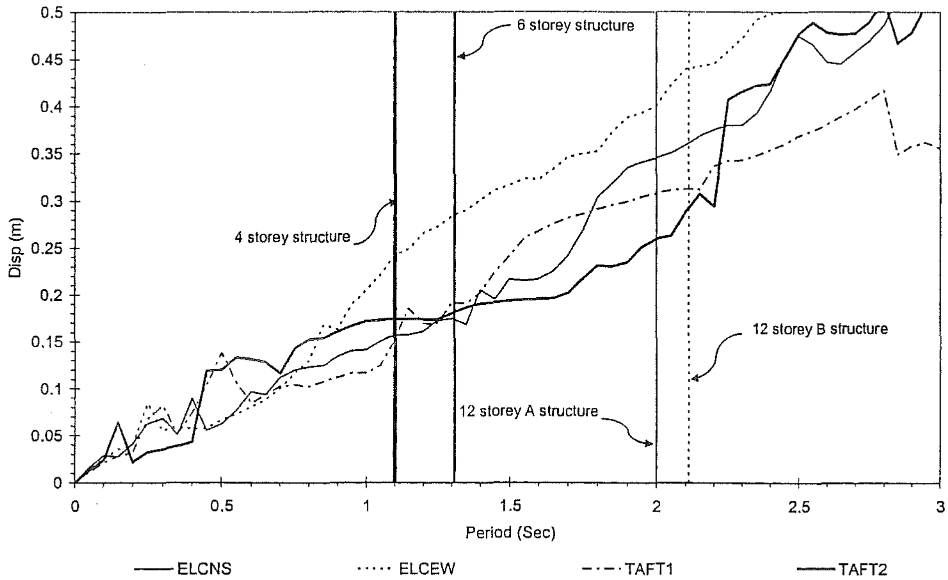



**Figure D-5 Inelastic displacement response spectra for modified ground acceleration records**

# APPENDIX E. COMPARISSON BETWEEN REDISTRIBUTED NOMINAL STRENGTHS OF BEAMS WHEN USING A MODAL OR AN EQUIVALENT STATIC ANALYSIS

A comparison is made between the beam redistributed nominal moments at column face coming from a modal analysis or an equivalent static analysis. In Tables E.1 through E.4, the redistributed nominal moment from the modal and equivalent static analyses for each level is given and then a ratio between these two values. On column 5 of each table, a ratio of the beam nominal moments at column face from earthquake alone between the modal values and the equivalent static values is given. Column 6 shows the same ratio but for P-delta actions.

From these tables, it can be seen that the difference between beam design moments when designing with an equivalent static or a modal analysis can be from 15% for the four storey structure. The difference between the two methods at any level increases with height in the building. For the six and twelve storey structures, the difference is on the order of 20% in the mid-height region of the structure. The difference in the calculation of P-delta forces after calculating the maximum displacements using a modal or an equivalent static analysis is not as marked in the lower levels of the structures where P-delta actions have the greater significance. The greatest difference arises when obtaining design forces from earthquake lateral loading giving approximately a 20% increase for all structures when designing through an equivalent static analysis.

**Table E-1 Four storey structure**

| *Level* | *Modal (kN m)* | *Equivalent static (kN m)* | *Static / Modal* | *Static / Modal (For earthquake induced forces only)* | *Static / Modal (For P-delta induced forces only)* |
|---|---|---|---|---|---|
| *1* | *345.5* | *385.4* | *1.12* | *1.16* | *1.00* |
| *2* | *303.8* | *351.2* | *1.16* | *1.20* | *1.03* |
| *3* | *215.0* | *236.8* | *1.10* | *1.20* | *1.07* |
| *4* | *92.3* | *106.2* | *1.15* | *1.16* | *1.10* |

**Table E-2 Six storey structure**

| *Level* | *Modal (kN m)* | *Equivalent static (kN m)* | *Static / Modal* | *Static / Modal (For earthquake induced forces only)* | *Static / Modal (For P-delta induced forces only)* |
|---|---|---|---|---|---|
| *1* | *464.3* | *531.2* | *1.14* | *1.19* | *1.03* |
| *2* | *452.3* | *536.6* | *1.19* | *1.24* | *1.07* |
| *3* | *372.4* | *453.5* | *1.22* | *1.25* | *1.12* |
| *4* | *286.2* | *345.6* | *1.21* | *1.22* | *1.16* |
| *5* | *192.8* | *226.3* | *1.17* | *1.17* | *1.20* |
| *6* | *89.0* | *101.0* | *1.13* | *1.13* | *1.24* |

**Table E-3 Twelve storey structure (Model A)**

| *Level* | *Modal (kN m)* | *Equivalent static (kN m)* | *Static / Modal* | *Static / Modal (For earthquake induced forces only)* | *Static / Modal (For P-delta induced forces only)* |
|---|---|---|---|---|---|
| *1* | *673.2* | *734.9* | *1.09* | *1.15* | *0.99* |
| *2* | *699.0* | *781.3* | *1.12* | *1.18* | *1.01* |
| *3* | *647.6* | *744.9* | *1.15* | *1.21* | *1.04* |
| *4* | *588.3* | *692.4* | *1.18* | *1.23* | *1.08* |
| *5* | *528.5* | *631.3* | *1.19* | *1.23* | *1.11* |
| *6* | *468.2* | *563.8* | *1.20* | *1.23* | *1.13* |
| *7* | *407.4* | *491.2* | *1.21* | *1.22* | *1.16* |
| *8* | *347.1* | *414.5* | *1.19* | *1.20* | *1.18* |
| *9* | *286.8* | *334.5* | *1.17* | *1.17* | *1.21* |
| *10* | *222.5* | *252.0* | *1.13* | *1.13* | *1.25* |
| *11* | *149.2* | *168.3* | *1.13* | *1.15* | *1.35* |
| *12* | *66.3* | *74.7* | *1.13* | *1.21* | *1.65* |

**Table E-4 Twelve storey structure (model B)**

| *Level* | *Modal (kN m)* | *Equivalent static (kN m)* | *Static / Modal* | *Static / Modal (For earthquake induced forces only)* | *Static / Modal (For P-delta induced forces only)* |
|---|---|---|---|---|---|
| *1* | *632.0* | *677.6* | *1.07* | *1.17* | *0.90* |
| *2* | *667.6* | *732.5* | *1.10* | *1.20* | *0.93* |
| *3* | *619.4* | *701.8* | *1.13* | *1.23* | *0.96* |
| *4* | *561.5* | *653.6* | *1.16* | *1.25* | *0.99* |
| *5* | *509.4* | *603.3* | *1.18* | *1.26* | *1.02* |
| *6* | *488.0* | *580.4* | *1.19* | *1.25* | *1.05* |
| *7* | *406.9* | *480.0* | *1.18* | *1.23* | *1.06* |
| *8* | *369.9* | *431.5* | *1.17* | *1.21* | *1.07* |
| *9* | *303.8* | *347.5* | *1.14* | *1.16* | *1.08* |
| *10* | *232.8* | *257.8* | *1.11* | *1.11* | *1.10* |
| *11* | *156.5* | *169.3* | *1.08* | *1.07* | *1.13* |
| *12* | *73.2* | *78.4* | *1.07* | *1.06* | *1.26* |

# APPENDIX F. RUAUMOKO INPUT FILES

## *F.1 Four Storey Structure*

```
FOUR STOREY, THREE BAY, CONCRETE MOMENT RESISTING FRAME

* UNITS kN, M
* SELF WEIGHT IS INCLUDED IN 4400 KN PER FLOOR (2200 kN PER FRAME)
* FLEXIBILITIES ADDED AT END OF MEMBERS TO ACCOUNT FOR JOINT DEFORMATIONS
*COLUMNS DESIGNED ACCORDING TO NZS 3101:1995
* UNIVERSITY OF CANTERBURY
* JOSE ANTONIO FLORES RUIZ


2 0 1 0 2 0 0 0 0 0 0
25 36 13 4 1 3 9.81 5.0 5.0 0.005 40 1
0 20 20 0 1 1 1 1 5 2 0 0
10 5 0.0001 0 0 0 0.5

NODES 0
1    0   0.0  1 1 1   0 0 0
2    7   0.0  1 1 1   0 0 0
3   14   0.0  1 1 1   0 0 0
4   21   0.0  1 1 1   0 0 0
5    0   3.4  0 0 0   6 0 0
6    7   3.4  0 0 0   0 0 0
7   14   3.4  0 0 0   6 0 0
8   21   3.4  0 0 0   6 0 0
9    0   6.8  0 0 0  10 0 0
10   7   6.8  0 0 0   0 0 0
11  14   6.8  0 0 0  10 0 0
12  21   6.8  0 0 0  10 0 0
13   0  10.2  0 0 0  14 0 0
14   7  10.2  0 0 0   0 0 0
15  14  10.2  0 0 0  14 0 0
16  21  10.2  0 0 0  14 0 0
17   0  13.6  0 0 0  18 0 0
18   7  13.6  0 0 0   0 0 0
19  14  13.6  0 0 0  18 0 0
20  21  13.6  0 0 0  18 0 0
21  28   0.0  1 1 1   0 0 0
22  28   3.4  0 0 1   0 0 0
23  28   6.8  0 0 1   0 0 0
24  28  10.2  0 0 1   0 0 0
25  28  13.6  0 0 1   0 0 0

DRIFT
4 8 12 16 20

ELEMENTS 0
1    1  5   6  0 0 0  !BEAM LEVEL 1
2    1  6   7  0 0 0
3    1  7   8  0 0 0
4    2  9  10  0 0 0  !BEAM LEVEL 2
5    2 10  11  0 0 0
6    2 11  12  0 0 0
7    3 13  14  0 0 0  !BEAM LEVEL 3
8    3 14  15  0 0 0
9    3 15  16  0 0 0
10   4 17  18  0 0 0  !BEAM LEVEL 4
11   4 18  19  0 0 0
12   4 19  20  0 0 0
13   5  1   5  0 0 0  !EXT COL GROUND LEVEL
14   6  5   9  0 0 0
15   7  9  13  0 0 0
16   8 13  17  0 0 0
17   9  2   6  0 0 0  !INT COL GROUND LEVEL
18  10  6  10  0 0 0
19  11 10  14  0 0 0
20  12 14  18  0 0 0
21   9  3   7  0 0 0  !INT COL GROUND LEVEL
22  10  7  11  0 0 0
23  11 11  15  0 0 0
24  12 15  19  0 0 0
25   5  4   8  0 0 0  !EXT COL GROUND LEVEL
26   6  8  12  0 0 0
27   7 12  16  0 0 0
28   8 16  20  0 0 0
29  13  8  22  0 0 0  !ADDITIONAL ELEMENTS FOR P-DELTA
30  13 12  23  0 0 0
31  13 16  24  0 0 0
32  13 20  25  0 0 0
33  13 21  22  0 0 0
34  13 22  23  0 0 0
35  13 23  24  0 0 0
36  13 24  25  0 0 0

PROPS

1 FRAME LEVEL1
1 0 0 4 0 0 0 0
2.5E7 1.04E7 0.3675 0.294 0.00585 0 0.3125 0.3125 1.0675e-6 1.0675e-6
0.007 0.007 0.35 0.35
1151.7 -7141.2 345.5 -345.5 345.5 -345.5
0.35 0.43 1 2

2 FRAME LEVEL2
1 0 0 4 0 0 0 0
2.5E7 1.04E7 0.3675 0.294 0.00585 0 0.3125 0.3125 1.0675e-6 1.0675e-6
0.007 0.007 0.35 0.35
```

```
1012.7 -7141.0 303.8 -303.8 303.8 -303.8
0.35 0.43 1 2

3 FRAME LEVEL3
1 0 0 4 0 0 0 0
2.5E7 1.04E7 0.3675 0.294 0.00585 0 0.3125 0.3125 1.0675e-6 1.0675e-6
0.007 0.007 0.35 0.35
716.7 -7140.7 215.0 -215.0 215.0 -215.0
0.35 0.43 1 2

4 FRAME LEVEL4
1 0 0 4 0 0 0 0
2.5E7 1.04E7 0.3675 0.294 0.00585 0 0.3125 0.3125 1.0675e-6 1.0675e-6
0.007 0.007 0.35 0.35
716.7 -7140.7 215.0 -215.0 215.0 -215.0
0.35 0.43 1 2

5 FRAME EXTCOLUMN1
2 0 0 4 0 0 3
2.5E7 1.04E7 0.3125 0.25 0.00407 0 0 0.35 0 1.7203E-6
0.007 0.007 0.375 0.375
-9528.8 -3738.9 1029.7 944.8 733.6 425.1 1560.0 1
-9048.8 -3738.9 903.7 818.8 608.9 298.9 1080.0
0.23 0.6 1 2

6 FRAME EXTCOL2
2 0 0 4 0 0 3
2.5E7 1.04E7 0.3125 0.25 0.00407 0 0.35 0.35 1.7203E-6 1.7203E-6
0.007 0.007 0.375 0.375
-9048.8 -3738.9 903.7 818.8 608.9 298.9 1080.0 1
-10458.8 -3738.9 1273.8 1188.9 976.1 668.8 2490.0
0.23 0.6 1 2

7 FRAME EXTCOL3
2 0 0 4 0 0 3
2.5E7 1.04E7 0.3125 0.25 0.00407 0 0.35 0.35 1.7203E-6 1.7203E-6
0.007 0.007 0.375 0.375
-10458.8 -3738.9 1273.8 1188.9 976.1 668.8 2490.0 1
-8748.8  -3738.9 824.9 740.0 530.9 219.4 780.0
0.23 0.6 1 2

8 FRAME EXTCOL4
2 0 0 4 0 0 3
2.5E7 1.04E7 0.3125 0.25 0.00407 0 0.35 0.35 1.7203E-6 1.7203E-6
0.007 0.007 0.375 0.375
-8748.8  -3738.9 824.9 740.0 530.9 219.4 780.0 1
-8328.8  -3738.9 714.7 629.8 421.8 107.1 360.0
0.23 0.6 1 2

9 FRAME INTCOLUMN1
2 0 0 4 0 0 3
2.5E7 1.04E7 0.3125 0.25 0.00407 0 0 0.35 0 1.7203E-6
0.007 0.007 0.375 0.375
-9318.8 -3738.9 974.6 889.7 679.0 370.0 1350.0 1
-9618.8 -3738.9 1053.3 968.4 756.9 449.1 1650.0
0.23 0.6 1 2

10 FRAME INTCOLUMN2
2 0 0 4 0 0 3
2.5E7 1.04E7 0.3125 0.25 0.00407 0 0.35 0.35 1.7203E-6 1.7203E-6
0.007 0.007 0.375 0.375
-9618.8 -3738.9 1053.3 968.4 756.9 449.1 1650.0 1
-11028.8 -3738.9 1423.4 1338.5 1124.7 818.3 3060.0
0.23 0.6 1 2

11 FRAME INTCOLUMN3
2 0 0 4 0 0 3
2.5E7 1.04E7 0.3125 0.25 0.00407 0 0.35 0.35 1.7203E-6 1.7203E-6
0.007 0.007 0.375 0.375
-11028.8 -3738.9 1423.4 1338.5 1124.7 818.3 3060.0 1
-9048.8  -3738.9  903.7  818.8  608.9 298.9 1080.0
0.23 0.6 1 2

12 FRAME INTCOLUMN4
2 0 0 4 0 0 3
2.5E7 1.04E7 0.3125 0.25 0.00407 0 0.35 0.35 1.7203E-6 1.7203E-6
0.007 0.007 0.375 0.375
-9048.8  -3738.9  903.7  818.8  608.9 298.9 1080.0 1
-8478.8  -3738.9  754.1  669.2  460.6 147.3  510.0
0.23 0.6 1 2

13 FRAME ADDITIONAL4PD
 2 3 0 0 0 0 0 0
 2.5E7 1.04E7 0.405 0.324 0.00615 0 0 0 0 0

WEIGHTS 0
1   0.0    0.0  0.0
2   0.0    0.0  0.0
3   0.0    0.0  0.0
4   0.0    0.0  0.0
5   550    0.0  0.0
6   550    0.0  0.0
7   550    0.0  0.0
8   550    0.0  0.0
9   550    0.0  0.0
10  550    0.0  0.0
11  550    0.0  0.0
12  550    0.0  0.0
13  550    0.0  0.0
14  550    0.0  0.0
15  550    0.0  0.0
16  550    0.0  0.0
```

```
17  550   0.0  0.0
18  550   0.0  0.0
19  550   0.0  0.0
20  550   0.0  0.0
21   0.0  0.0  0.0
22   0.0  0.0  0.0
23   0.0  0.0  0.0
24   0.0  0.0  0.0
25   0.0  0.0  0.0
```

```
LOADS
1      0.00        0.00   0.00
2      0.00        0.00   0.00
3      0.00        0.00   0.00
4      0.00        0.00   0.00
5      0.00      -50.06   0.00
6      0.00      -80.28   0.00
7      0.00      -80.28   0.00
8      0.00      -50.06   0.00
9      0.00      -50.06   0.00
10     0.00      -80.28   0.00
11     0.00      -80.28   0.00
12     0.00      -50.06   0.00
13     0.00      -50.06   0.00
14     0.00      -80.28   0.00
15     0.00      -80.28   0.00
16     0.00      -50.06   0.00
17     0.00      -40.14   0.00
18     0.00      -70.37   0.00
19     0.00      -70.37   0.00
20     0.00      -40.14   0.00
21     0.00        0.00   0.00
22     0.00    -1939.33   0.00
23     0.00    -1939.33   0.00
24     0.00    -1939.33   0.00
25     0.00    -1978.98   0.00
```

```
EQUAKE I:\quake\4203ELC1.EQF
3 1 0.02 0.8333 27.10 0 0 1
```

## *F.2 Six Storey Structure*

```
SIX STOREY, THREE BAY, CONCRETE MOMENT RESISTING FRAME

* UNITS KN, M
* SELF WEIGHT IS INCLUDED IN 4400 KN PER FLOOR (2200 kN PER FRAME)
* FLEXIBILITIES ADDED AT END OF MEMBERS TO ACCOUNT FOR JOINT DEFORMATIONS
*COLUMNS DESIGNED ACCORDING TO NZS 3101:1995
* UNIVERSITY OF CANTERBURY
* JOSE ANTONIO FLORES RUIZ

2 0 1 0 2 0 0 0 0
35 54 19 6 1 3 9.81 5.0 5.0 0.005 40 1
20 20 20 0  1 1 2 2 7 2 1
10 5 0.0001 0 0 0

NODES 0
1   0.0   0.00   1 1 1    0 0 0
2   7.0   0.00   1 1 1    0 0 0
3  14.0   0.00   1 1 1    0 0 0
4  21.0   0.00   1 1 1    0 0 0
5   0.0   3.40   0 0 0    6 0 0
6   7.0   3.40   0 0 0    0 0 0
7  14.0   3.40   0 0 0    6 0 0
8  21.0   3.40   0 0 0    6 0 0
9   0.0   6.80   0 0 0   10 0 0
10  7.0   6.80   0 0 0    0 0 0
11 14.0   6.80   0 0 0   10 0 0
12 21.0   6.80   0 0 0   10 0 0
13  0.0  10.20   0 0 0   14 0 0
14  7.0  10.20   0 0 0    0 0 0
15 14.0  10.20   0 0 0   14 0 0
16 21.0  10.20   0 0 0   14 0 0
17  0.0  13.60   0 0 0   18 0 0
18  7.0  13.60   0 0 0    0 0 0
19 14.0  13.60   0 0 0   18 0 0
20 21.0  13.60   0 0 0   18 0 0
21  0.0  17.00   0 0 0   22 0 0
22  7.0  17.00   0 0 0    0 0 0
23 14.0  17.00   0 0 0   22 0 0
24 21.0  17.00   0 0 0   22 0 0
25  0.0  20.40   0 0 0   26 0 0
26  7.0  20.40   0 0 0    0 0 0
27 14.0  20.40   0 0 0   26 0 0
28 21.0  20.40   0 0 0   26 0 0
29 28.0   0.00   1 1 1    0 0 0
30 28.0   3.40   0 0 1    0 0 0
31 28.0   6.80   0 0 1    0 0 0
32 28.0  10.20   0 0 1    0 0 0
33 28.0  13.60   0 0 1    0 0 0
34 28.0  17.00   0 0 1    0 0 0
35 28.0  20.40   0 0 1    0 0 0

DRIFT
4 8 12 16 20 24 28

ELEMENTS 0
1   1   5   6  0 0 0        !BEAMS LEVEL 1
2   1   6   7  0 0 0
3   1   7   8  0 0 0
```

```
4  2   9 10 0 0 0     !BEAM LEVEL 2
5  2  10 11 0 0 0
6  2  11 12 0 0 0
7  3  13 14 0 0 0     !BEAM LEVEL 3
8  3  14 15 0 0 0
9  3  15 16 0 0 0
10 4  17 18 0 0 0     !BEAM LEVEL 4
11 4  18 19 0 0 0
12 4  19 20 0 0 0
13 5  21 22 0 0 0     !BEAM LEVEL 5
14 5  22 23 0 0 0
15 5  23 24 0 0 0
16 6  25 26 0 0 0     !BEAM LEVEL 6
17 6  26 27 0 0 0
18 6  27 28 0 0 0
19 7   1  5 0 0 0     !EXT COL LEVEL 1
20 8   5  9 0 0 0
21 9   9 13 0 0 0
22 10 13 17 0 0 0
23 11 17 21 0 0 0
24 12 21 25 0 0 0
25 13  2  6 0 0 0     !INT COL LEVEL 1
26 14  6 10 0 0 0
27 15 10 14 0 0 0
28 16 14 18 0 0 0
29 17 18 22 0 0 0
30 18 22 26 0 0 0
31 13  3  7 0 0 0     !INT COL LEVEL 1
32 14  7 11 0 0 0
33 15 11 15 0 0 0
34 16 15 19 0 0 0
35 17 19 23 0 0 0
36 18 23 27 0 0 0
37  7  4  8 0 0 0     !EXT COL LEVEL 1
38  8  8 12 0 0 0
39  9 12 16 0 0 0
40 10 16 20 0 0 0
41 11 20 24 0 0 0
42 12 24 28 0 0 0
43 19 29 30 0 0 0     ! ADDITIONAL ELEMENTS FOR P-DELTA
44 19 30 31 0 0 0
45 19 31 32 0 0 0
46 19 32 33 0 0 0
47 19 33 34 0 0 0
48 19 34 35 0 0 0
49 19  8 30 0 0 0
50 19 12 31 0 0 0
51 19 16 32 0 0 0
52 19 20 33 0 0 0
53 19 24 34 0 0 0
54 19 28 35 0 0 0
```

```
PROPS
 1 FRAME BEAMLEV1
  1 0 0 4 0 0 3
  2.5E7 1.04E7 0.43625 0.349 0.00923 0 0.3375 0.3375 7.3131E-7 7.3131E-7
  0.007 0.007 0.3875 0.3875
  1375.7 -8894.5 464.3 -464.3 464.3 -464.3
  0.35 0.43 1 2

 2 FRAME BEAMLEV2
  1 0 0 4 0 0 3
  2.5E7 1.04E7 0.436250 0.349 0.00923 0 0.3375 0.3375 7.3131E-7 7.3131E-7
  0.007 0.007 0.3875 0.3875
  1340.1 -8894.5 452.3 -452.3 452.3 -452.3
  0.35 0.43 1 2

 3 FRAME BEAMLEV3
  1 0 0 4 0 0 3
  2.5E7 1.04E7 0.436250 0.349 0.00923 0 0.3375 0.3375 7.3131E-7 7.3131E-7
  0.007 0.007 0.3875 0.3875
  1103.4 -8894.2 372.4 -372.4 372.4 -372.4
  0.35 0.43 1 2

 4 FRAME BEAMLEV4
  1 0 0 4 0 0 3
  2.5E7 1.04E7 0.436250 0.349 0.00923 0 0.3375 0.3375 7.3131E-7 7.3131E-7
  0.007 0.007 0.3875 0.3875
  1103.4 -8894.2 372.4 -372.4 372.4 -372.4
  0.35 0.43 1 2

 5 FRAME BEAMLEV5
  1 0 0 4 0 0 3
  2.5E7 1.04E7 0.436250 0.349 0.00923 0 0.3375 0.3375 7.3131E-7 7.3131E-7
  0.007 0.007 0.3875 0.3875
  1103.4 -8894.2 372.4 -372.4 372.4 -372.4
  0.35 0.43 1 2

 6 FRAME BEAMLEV6
  1 0 0 4 0 0 3
  2.5E7 1.04E7 0.436250 0.349 0.00923 0 0.3375 0.3375 7.3131E-7 7.3131E-7
  0.007 0.007 0.3875 0.3875
  1103.4 -8894.2 372.4 -372.4 372.4 -372.4
  0.35 0.43 1 2


 7 FRAME EXTGLCOLUMN
  2 0 0 4 0 0 3 0
  2.5E7 1.04E7 0.405 0.324 0.00615 0 0 0.3875 0 1.2601E-6
  0.007 0.007 0.3375 0.3375
  -12457.5 -4876.9 1481.1 1364.2 1069.6 631.2 2130.0 1
  -11857.5 -4876.9 1308.6 1191.7 898.6 458.6 1530.0
```

0.23 0.6 1 2

8 FRAME EXTCOLUMNlev2
2 0 0 4 0 0 3 0
2.5E7 1.04E7 0.405 0.324 0.00615 0 0.3875 0.3875 1.2601E-6 1.2601E-6
0.007 0.007 0.3375 0.3375
-11857.5 -4876.9 1308.6 1191.7 898.6 458.6 1530.0 1
-12157.5 -4876.9 1394.8 1278.0 984.3 544.7 1830.0
0.23 0.6 1 2

9 FRAME EXTCOLUMNlev3
2 0 0 4 0 0 3 0
2.5E7 1.04E7 0.405 0.324 0.00615 0 0.3875 0.3875 1.2601E-6 1.2601E-6
0.007 0.007 0.3375 0.3375
-12157.5 -4876.9 1394.8 1278.0 984.3 544.7 1830.0 1
-11947.5 -4876.9 1334.4 1217.6 924.2 484.5 1620.0
0.23 0.6 1 2

10 FRAME EXTCOLUMNlev4
2 0 0 4 0 0 3 0
2.5E7 1.04E7 0.405 0.324 0.00615 0 0.3875 0.3875 1.2601E-6 1.2601E-6
0.007 0.007 0.3375 0.3375
-11947.5 -4876.9 1334.4 1217.6 924.2 484.5 1620.0 1
-12037.5 -4876.9 1360.3 1243.5 949.9 510.9 1710.0
0.23 0.6 1 2

11 FRAME EXTCOLUMNlev5
2 0 0 4 0 0 3 0
2.5E7 1.04E7 0.405 0.324 0.00615 0 0.3875 0.3875 1.2601E-6 1.2601E-6
0.007 0.007 0.3375 0.3375
-12037.5 -4876.9 1360.3 1243.5 949.9 510.9 1710.0 1
-11857.5 -4876.9 1308.6 1191.7 898.6 458.6 1530.0
0.23 0.6 1 2

12 FRAME EXTCOLUMNlev6
2 0 0 4 0 0 3 0
2.5E7 1.04E7 0.405 0.324 0.00615 0 0.3875 0.3875 1.2601E-6 1.2601E-6
0.007 0.007 0.3375 0.3375
-11857.5 -4876.9 1308.6 1191.7 898.6 458.6 1530.0 1
-11302.5 -4876.9 1149.0 1032.2 740.3 298.5 975.0
0.23 0.6 1 2

13 FRAME INTGLCOLUMN
2 0 0 4 0 0 3 0
2.5E7 1.04E7 0.405 0.324 0.00615 0 0 0.3875 0 1.2601E-6
0.007 0.007 0.3375 0.3375
-11737.5 -4876.9 1274.1 1157.2 864.3 423.9 1410.0 1
-12397.5 -4876.9 1463.8 1347.0 1052.8 614.4 2070.0
0.23 0.6 1 2

14 FRAME INTCOLUMNlev1
2 0 0 4 0 0 3 0
2.5E7 1.04E7 0.405 0.324 0.00615 0 0.3875 0.3875 1.2601E-6 1.2601E-6
0.007 0.007 0.3375 0.3375
-12397.5 -4876.9 1463.8 1347.0 1052.8 614.4 2070.0 1
-12847.5 -4876.9 1593.2 1476.4 1181.5 743.5 2520.0
0.23 0.6 1 2

15 FRAME INTCOLUMNlev2
2 0 0 4 0 0 3 0
2.5E7 1.04E7 0.405 0.324 0.00615 0 0.3875 0.3875 1.2601E-6 1.2601E-6
0.007 0.007 0.3375 0.3375
-12847.5 -4876.9 1593.2 1476.4 1181.5 743.5 2520.0 1
-12517.5 -4876.9 1498.3 1381.5 1087.4 648.1 2190.0
0.23 0.6 1 2

16 FRAME INTCOLUMNlev3
2 0 0 4 0 0 3 0
2.5E7 1.04E7 0.405 0.324 0.00615 0 0.3875 0.3875 1.2601E-6 1.2601E-6
0.007 0.007 0.3375 0.3375
-12517.5 -4876.9 1498.3 1381.5 1087.4 648.1 2190.0 1
-12817.5 -4876.9 1584.6 1467.7 1173.2 734.7 2490.0
0.23 0.6 1 2

17 FRAME INTCOLUMNlev4
2 0 0 4 0 0 3 0
2.5E7 1.04E7 0.405 0.324 0.00615 0 0.3875 0.3875 1.2601E-6 1.2601E-6
0.007 0.007 0.3375 0.3375
-12817.5 -4876.9 1584.6 1467.7 1173.2 734.7 2490.0 1
-12547.5 -4876.9 1506.9 1390.1 1095.7 656.7 2220.0
0.23 0.6 1 2

18 FRAME INTCOLUMNlev5
2 0 0 4 0 0 3 0
2.5E7 1.04E7 0.405 0.324 0.00615 0 0.3875 0.3875 1.2601E-6 1.2601E-6
0.007 0.007 0.3375 0.3375
-12547.5 -4876.9 1506.9 1390.1 1095.7 656.7 2220.0 1
-11302.5 -4876.9 1149.0 1032.2 740.3 298.5 975.0
0.23 0.6 1 2

19 FRAME ADDITIONAL4PD
2 3 0 0 0 0 0 0
2.5E7 1.04E7 0.405 0.324 0.00615 0 0 0 0 0

WEIGHTS 0
1 0.0 0.0 0.0
2 0.0 0.0 0.0
3 0.0 0.0 0.0
4 0.0 0.0 0.0
5 550 0.0 0.0
6 550 0.0 0.0
7 550 0.0 0.0

```
8   550   0.0  0.0
9   550   0.0  0.0
10  550   0.0  0.0
11  550   0.0  0.0
12  550   0.0  0.0
13  550   0.0  0.0
14  550   0.0  0.0
15  550   0.0  0.0
16  550   0.0  0.0
17  550   0.0  0.0
18  550   0.0  0.0
19  550   0.0  0.0
20  550   0.0  0.0
21  550   0.0  0.0
22  550   0.0  0.0
23  550   0.0  0.0
24  550   0.0  0.0
25  550   0.0  0.0
26  550   0.0  0.0
27  550   0.0  0.0
28  550   0.0  0.0
35    0.0   0.0  0.0
```

```
LOADS
 1  0.0      0.0     0.0
 2  0.0      0.0     0.0
 3  0.0      0.0     0.0
 4  0.0      0.0     0.0
 5  0.0   -56.68     0.0
 6  0.0   -93.50     0.0
 7  0.0   -93.50     0.0
 8  0.0   -56.68     0.0
 9  0.0   -56.68     0.0
10  0.0   -93.50     0.0
11  0.0   -93.50     0.0
12  0.0   -56.68     0.0
13  0.0   -56.68     0.0
14  0.0   -93.50     0.0
15  0.0   -93.50     0.0
16  0.0   -56.68     0.0
17  0.0   -56.68     0.0
18  0.0   -93.50     0.0
19  0.0      -93.50     0.0
20  0.0      -56.68     0.0
21  0.0      -56.68     0.0
22  0.0      -93.50     0.0
23  0.0      -93.50     0.0
24  0.0      -56.68     0.0
25  0.0      -46.75     0.0
26  0.0      -83.57     0.0
27  0.0      -83.57     0.0
28  0.0      -46.75     0.0
29  0.0        0.00     0.0
30  0.0   -1899.62     0.0
31  0.0   -1899.62     0.0
32  0.0   -1899.62     0.0
33  0.0   -1899.62     0.0
34  0.0   -1899.62     0.0
35  0.0   -1939.35     0.0
```

```
EQUAKE L:\QUAKE\4203ELC1.EQF
3 1 0.02 0.83333 27.10 0 0 1
```

## *F.3 Twelve Storey Structure A*

```
TWELVE STOREY A, THREE BAY, CONCRETE MOMENT RESISTING FRAME

* UNITS kN, M
* SELF WEIGHT IS INCLUDED IN 4400 KN PER FLOOR (2200 kN PER FRAME)
* FLEXIBILITIES ADDED AT END OF MEMBERS TO ACCOUNT FOR JOINT DEFORMATIONS
*COLUMNS DESIGNED ACCORDING TO NZS 3101:1995
* UNIVERSITY OF CANTERBURY
* JOSE ANTONIO FLORES RUIZ

2 0 1 0 2 0 0 0 0 0 0
65 108 37 6 1 3 9.81 5.0 5.0 0.005 40 1
20 20 20 0  2 2 1 1 13 2 0
10 5 0.0001 0 0 0

NODES 0
 1    0    0.0  1 1 1    0 0 0
 2    7    0.0  1 1 1    0 0 0
 3   14    0.0  1 1 1    0 0 0
 4   21    0.0  1 1 1    0 0 0
 5    0    3.4  0 0 0    6 0 0
 6    7    3.4  0 0 0    0 0 0
 7   14    3.4  0 0 0    6 0 0
 8   21    3.4  0 0 0    6 0 0
 9    0    6.8  0 0 0   10 0 0
10    7    6.8  0 0 0    0 0 0
11   14    6.8  0 0 0   10 0 0
12   21    6.8  0 0 0   10 0 0
13    0   10.2  0 0 0   14 0 0
14    7   10.2  0 0 0    0 0 0
15   14   10.2  0 0 0   14 0 0
16   21   10.2  0 0 0   14 0 0
17    0   13.6  0 0 0   18 0 0
18    7   13.6  0 0 0    0 0 0
19   14   13.6  0 0 0   18 0 0
20   21   13.6  0 0 0   18 0 0
21    0   17.0  0 0 0   22 0 0
22    7   17.0  0 0 0    0 0 0
```

```
23  14  17.0  0 0 0  22  0 0
24  21  17.0  0 0 0  22  0 0
25   0  20.4  0 0 0  26  0 0
26   7  20.4  0 0 0   0  0 0
27  14  20.4  0 0 0  26  0 0
28  21  20.4  0 0 0  26  0 0
29   0  23.8  0 0 0  30  0 0
30   7  23.8  0 0 0   0  0 0
31  14  23.8  0 0 0  30  0 0
32  21  23.8  0 0 0  30  0 0
33   0  27.2  0 0 0  34  0 0
34   7  27.2  0 0 0   0  0 0
35  14  27.2  0 0 0  34  0 0
36  21  27.2  0 0 0  34  0 0
37   0  30.6  0 0 0  38  0 0
38   7  30.6  0 0 0   0  0 0
39  14  30.6  0 0 0  38  0 0
40  21  30.6  0 0 0  38  0 0
41   0  34.0  0 0 0  42  0 0
42   7  34.0  0 0 0   0  0 0
43  14  34.0  0 0 0  42  0 0
44  21  34.0  0 0 0  42  0 0
45   0  37.4  0 0 0  46  0 0
46   7  37.4  0 0 0   0  0 0
47  14  37.4  0 0 0  46  0 0
48  21  37.4  0 0 0  46  0 0
49   0  40.8  0 0 0  50  0 0
50   7  40.8  0 0 0   0  0 0
51  14  40.8  0 0 0  50  0 0
52  21  40.8  0 0 0  50  0 0
53  28   0.0  1 1 1   0  0 0
54  28   3.4  0 0 1   0  0 0
55  28   6.8  0 0 1   0  0 0
56  28  10.2  0 0 1   0  0 0
57  28  13.6  0 0 1   0  0 0
58  28  17.0  0 0 1   0  0 0
59  28  20.4  0 0 1   0  0 0
60  28  23.8  0 0 1   0  0 0
61  28  27.2  0 0 1   0  0 0
62  28  30.6  0 0 1   0  0 0
63  28  34.0  0 0 1   0  0 0
64  28  37.4  0 0 1   0  0 0
65  28  40.8  0 0 1   0  0 0

DRIFT
4 8 12 16 20 24 28 32 36 40 44 48 52

ELEMENTS 0
 1   1   5   6  0 0 0          !BEAM LEVEL 1
 2   1   6   7  0 0 0
 3   1   7   8  0 0 0
 4   2   9  10  0 0 0          !BEAM LEVEL 2
 5   2  10  11  0 0 0
 6   2  11  12  0 0 0
 7   3  13  14  0 0 0          !BEAM LEVEL 3
 8   3  14  15  0 0 0
 9   3  15  16  0 0 0
10   4  17  18  0 0 0          !BEAM LEVEL 4
11   4  18  19  0 0 0
12   4  19  20  0 0 0
13   5  21  22  0 0 0          !BEAM LEVEL 5
14   5  22  23  0 0 0
15   5  23  24  0 0 0
16   6  25  26  0 0 0          !BEAM LEVEL 6
17   6  26  27  0 0 0
18   6  27  28  0 0 0
19   7  29  30  0 0 0          !BEAM LEVEL 7
20   7  30  31  0 0 0
21   7  31  32  0 0 0
22   8  33  34  0 0 0          !BEAM LEVEL 8
23   8  34  35  0 0 0
24   8  35  36  0 0 0
25   9  37  38  0 0 0          !BEAM LEVEL 9
26   9  38  39  0 0 0
27   9  39  40  0 0 0
28  10  41  42  0 0 0          !BEAM LEVEL 10
29  10  42  43  0 0 0
30  10  43  44  0 0 0
31  11  45  46  0 0 0          !BEAM LEVEL 11
32  11  46  47  0 0 0
33  11  47  48  0 0 0
34  12  49  50  0 0 0          !BEAM LEVEL 12
35  12  50  51  0 0 0
36  12  51  52  0 0 0
37  13   1   5  0 0 0          !EXT COL GROUND LEVEL
38  14   5   9  0 0 0
39  15   9  13  0 0 0
40  16  13  17  0 0 0
41  17  17  21  0 0 0
42  18  21  25  0 0 0
43  19  25  29  0 0 0
44  20  29  33  0 0 0
45  21  33  37  0 0 0
46  22  37  41  0 0 0
47  23  41  45  0 0 0
48  24  45  49  0 0 0
49  25   2   6  0 0 0          !INT COL GROUND LEVEL
50  26   6  10  0 0 0
51  27  10  14  0 0 0
52  28  14  18  0 0 0
53  29  18  22  0 0 0
54  30  22  26  0 0 0
```

```
55   31 26 30  0 0 0
56   32 30 34  0 0 0
57   33 34 38  0 0 0
58   34 38 42  0 0 0
59   35 42 46  0 0 0
60   36 46 50  0 0 0
61   25  3  7  0 0 0          !INT COL GROUND LEVEL
62   26  7 11  0 0 0
63   27  11 15  0 0 0
64   28  15 19  0 0 0
65   29  19 23  0 0 0
66   30  23 27  0 0 0
67   31  27 31  0 0 0
68   32  31 35  0 0 0
69   33  35 39  0 0 0
70   34  39 43  0 0 0
71   35  43 47  0 0 0
72   36  47 51  0 0 0
73   13   4  8  0 0 0         !EXT COL GROUND LEVEL
74   14   8 12  0 0 0
75   15  12 16  0 0 0
76   16  16 20  0 0 0
77   17  20 24  0 0 0
78   18  24 28  0 0 0
79   19  28 32  0 0 0
80   20  32 36  0 0 0
81   21  36 40  0 0 0
82   22  40 44  0 0 0
83   23  44 48  0 0 0
84   24  48 52  0 0 0
85   37   8 54  0 0 0         !ADDITIONAL ELEMENTS FOR P-DELTA
86   37  12 55  0 0 0
87   37  16 56  0 0 0
88   37  20 57  0 0 0
89   37  24 58  0 0 0
90   37  28 59  0 0 0
91   37  32 60  0 0 0
92   37  36 61  0 0 0
93   37  40 62  0 0 0
94   37  44 63  0 0 0
95   37  48 64  0 0 0
96   37  52 65  0 0 0
97   37  53 54  0 0 0
98   37  54 55  0 0 0
99   37  55 56  0 0 0
100  37  56 57  0 0 0
101  37  57 58  0 0 0
102  37  58 59  0 0 0
103  37  59 60  0 0 0
104  37  60 61  0 0 0
105  37  61 62  0 0 0
106  37  62 63  0 0 0
107  37  63 64  0 0 0
108  37  64 65  0 0 0


PROPS
1 FRAME LEVEL1
 1 0 0 4 0 0 3 0
 2.5E7 1.04E7 0.5825 0.466 0.017 0 0.375 0.375 4.4157e-7 4.4157e-7
 0.004 0.004 0.45 0.45
 1683.0 -12624.2 673.2 -673.2 673.2 -673.2
 0.35 0.43 1 2

2 FRAME LEVEL2
 1 0 0 4 0 0 3 0
 2.5E7 1.04E7 0.5825 0.466 0.017 0 0.375 0.375 4.4157e-7 4.4157e-7
 0.004 0.004 0.45 0.45
 1747.5 -12624.2 699.0 -699.0 699.0 -699.0
 0.35 0.43 1 2

3 FRAME LEVEL3
 1 0 0 4 0 0 3 0
 2.5E7 1.04E7 0.5825 0.466 0.017 0 0.375 0.375 4.4157e-7 4.4157e-7
 0.004 0.004 0.45 0.45
 1619.0 -12624.1 647.6 -647.6 647.6 -647.6
 0.35 0.43 1 2

4 FRAME LEVEL4
 1 0 0 4 0 0 3 0
 2.5E7 1.04E7 0.5825 0.466 0.017 0 0.375 0.375 4.4157e-7 4.4157e-7
 0.004 0.004 0.45 0.45
 1470.8 -12624.0 588.3 -588.3 588.3 -588.3
 0.35 0.43 1 2

5 FRAME LEVEL5
 1 0 0 4 0 0 3 0
 2.5E7 1.04E7 0.5825 0.466 0.017 0 0.375 0.375 4.4157e-7 4.4157e-7
 0.004 0.004 0.45 0.45
 1321.3 -12623.8 528.5 -528.5 528.5 -528.5
 0.35 0.43 1 2

6 FRAME LEVEL6
 1 0 0 4 0 0 3 0
 2.5E7 1.04E7 0.5825 0.466 0.017 0 0.375 0.375 4.4157e-7 4.4157e-7
 0.004 0.004 0.45 0.45
 1321.3 -12623.8 528.5 -528.5 528.5 -528.5
 0.35 0.43 1 2

7 FRAME LEVEL7
 1 0 0 4 0 0 3 0
```

```
2.5E7 1.04E7 0.5825 0.466 0.017 0 0.375 0.375 4.4157e-7 4.4157e-7
0.004 0.004 0.45 0.45
1321.3 -12623.8 528.5 -528.5 528.5 -528.5
0.35 0.43 1 2

8 FRAME LEVEL8
1 0 0 4 0 0 3 0
2.5E7 1.04E7 0.5825 0.466 0.017 0 0.375 0.375 4.4157e-7 4.4157e-7
0.004 0.004 0.45 0.45
1321.3 -12623.8 528.5 -528.5 528.5 -528.5
0.35 0.43 1 2

9 FRAME LEVEL9
1 0 0 4 0 0 3 0
2.5E7 1.04E7 0.5825 0.466 0.017 0 0.375 0.375 4.4157e-7 4.4157e-7
0.004 0.004 0.45 0.45
1321.3 -12623.8 528.5 -528.5 528.5 -528.5
0.35 0.43 1 2

10 FRAME LEVEL10
1 0 0 4 0 0 3 0
2.5E7 1.04E7 0.5825 0.466 0.017 0 0.375 0.375 4.4157e-7 4.4157e-7
0.004 0.004 0.45 0.45
1321.3 -12623.8 528.5 -528.5 528.5 -528.5
0.35 0.43 1 2

11 FRAME LEVEL11
1 0 0 4 0 0 3 0
2.5E7 1.04E7 0.5825 0.466 0.017 0 0.375 0.375 4.4157e-7 4.4157e-7
0.004 0.004 0.45 0.45
1321.3 -12623.8 528.5 -528.5 528.5 -528.5
0.35 0.43 1 2

12 FRAME LEVEL12
1 0 0 4 0 0 3 0
2.5E7 1.04E7 0.5825 0.466 0.017 0 0.375 0.375 4.4157e-7 4.4157e-7
0.004 0.004 0.45 0.45
1321.3 -12623.8 528.5 -528.5 528.5 -528.5
0.35 0.43 1 2

13 FRAME EXTCOLUMNSATBASE
2 0 0 4 0 0 3
2.5E7 1.04E7 0.4875 0.39 0.00914 0 0 0.45 0 9.8462e-7
0.004 0.004 0.375 0.375
-15401.3 -5917.3 2128.0 1975.1 1584.9 985.8 2970.0 1
-14771.3 -5917.3 1923.2 1770.4 1380.8 780.7 2340.0
0.23 0.6 1 2

14 FRAME EXTCOLUMN2
2 0 0 4 0 0 3
2.5E7 1.04E7 0.4875 0.39 0.00914 0 0.45 0.45 9.8462e-7 9.8462e-7
0.004 0.004 0.375 0.375
-14771.3 -5917.3 1923.2 1770.4 1380.8 780.7 2340.0 1
-15041.3 -5917.3 2011.0 1858.1 1468.4 868.2 2610.0
0.23 0.6 1 2

15 FRAME EXTCOLUMN3
2 0 0 4 0 0 3
2.5E7 1.04E7 0.4875 0.39 0.00914 0 0.45 0.45 9.8462e-7 9.8462e-7
0.004 0.004 0.375 0.375
-15041.3 -5917.3 2011.0 1858.1 1468.4 868.2 2610.0 1
-15101.3 -5917.3 2030.5 1877.6 1487.6 888.6 2670.0
0.23 0.6 1 2

16 FRAME EXTCOLUMN4
2 0 0 4 0 0 3
2.5E7 1.04E7 0.4875 0.39 0.00914 0 0.45 0.45 9.8462e-7 9.8462e-7
0.004 0.004 0.375 0.375
-15101.3 -5917.3 2030.5 1877.6 1487.6 888.6 2670.0 1
-15011.3 -5917.3 2001.2 1848.4 1458.2 859.3 2580.0
0.23 0.6 1 2

17 FRAME EXTCOLUMN5
2 0 0 4 0 0 3
2.5E7 1.04E7 0.4875 0.39 0.00914 0 0.45 0.45 9.8462e-7 9.8462e-7
0.004 0.004 0.375 0.375
-15011.3 -5917.3 2001.2 1848.4 1458.2 859.3 2580.0  1
-14771.3 -5917.3 1923.2 1770.4 1380.8 780.7 2340.0
0.23 0.6 1 2

18 FRAME EXTCOLUMN6
2 0 0 4 0 0 3
2.5E7 1.04E7 0.4875 0.39 0.00914 0 0.45 0.45 9.8462e-7 9.8462e-7
0.004 0.004 0.375 0.375
-14771.3 -5917.3 1923.2 1770.4 1380.8 780.7 2340.0 1
-14741.3 -5917.3 1913.5 1760.6 1370.7 770.8 2310.0
0.23 0.6 1 2

19 FRAME EXTCOLUMN7
2 0 0 4 0 0 3
2.5E7 1.04E7 0.4875 0.39 0.00914 0 0.45 0.45 9.8462e-7 9.8462e-7
0.004 0.004 0.375 0.375
-14741.3 -5917.3 1913.5 1760.6 1370.7 770.8 2310.0 1
-14741.3 -5917.3 1913.5 1760.6 1370.7 770.8 2310.0
0.23 0.6 1 2

20 FRAME EXTCOLUMN8
2 0 0 4 0 0 3
2.5E7 1.04E7 0.4875 0.39 0.00914 0 0.45 0.45 9.8462e-7 9.8462e-7
0.004 0.004 0.375 0.375
-14741.3 -5917.3 1913.5 1760.6 1370.7 770.8 2310.0 1
-14711.3 -5917.3 1903.7 1750.9 1361.0 761.0 2280.0
```

0.23 0.6 1 2

21 FRAME EXTCOLUMN9
2 0 0 4 0 0 3
2.5E7 1.04E7 0.4875 0.39 0.00914 0 0.45 0.45 9.8462e-7 9.8462e-7
0.004 0.004 0.375 0.375
-14711.3 -5917.3 1903.7 1750.9 1361.0 761.0 2280.0 1
-14741.3 -5917.3 1913.5 1760.6 1370.7 770.8 2310.0
0.23 0.6 1 2

22 FRAME EXTCOLUMN10
2 0 0 4 0 0 3
2.5E7 1.04E7 0.4875 0.39 0.00914 0 0.45 0.45 9.8462e-7 9.8462e-7
0.004 0.004 0.375 0.375
-14741.3 -5917.3 1913.5 1760.6 1370.7 770.8 2310.0 1
-14801.3 -5917.3 1933.0 1780.1 1390.4 790.5 2370.0
0.23 0.6 1 2

23 FRAME EXTCOLUMN11
2 0 0 4 0 0 3
2.5E7 1.04E7 0.4875 0.39 0.00914 0 0.45 0.45 9.8462e-7 9.8462e-7
0.004 0.004 0.375 0.375
-14801.3 -5917.3 1933.0 1780.1 1390.4 790.5 2370.0 1
-14351.3 -5917.3 1786.7 1633.9 1244.2 644.0 1920.0
0.23 0.6 1 2

24 FRAME EXTCOLUMN12
2 0 0 4 0 0 3
2.5E7 1.04E7 0.4875 0.39 0.00914 0 0.45 0.45 9.8462e-7 9.8462e-7
0.004 0.004 0.375 0.375
-14351.3 -5917.3 1786.7 1633.9 1244.2 644.0 1920.0 1
-13601.3 -5917.3 1543.0 1390.1 1001.4 400.3 1170.0
0.23 0.6 1 2

25 FRAME INTCOLUMNSATBASE
2 0 0 4 0 0 3
2.5E7 1.04E7 0.4875 0.39 0.00914 0 0 0.45 0 9.8462e-7
0.004 0.004 0.375 0.375
-13601.3 -5917.3 1543.0 1390.1 1001.4 400.3 1170.0 1
-14111.3 -5917.3 1708.7 1555.9 1166.4 566.5 1680.0
0.23 0.6 1 2

26 FRAME INTCOLUMN2
2 0 0 4 0 0 3
2.5E7 1.04E7 0.4875 0.39 0.00914 0 0.45 0.45 9.8462e-7 9.8462e-7
0.004 0.004 0.375 0.375
-14111.3 -5917.3 1708.7 1555.9 1166.4 566.5 1680.0 1
-14441.3 -5917.3 1816.0 1663.1 1273.5 673.3 2010.0
0.23 0.6 1 2

27 FRAME INTCOLUMN3
2 0 0 4 0 0 3
2.5E7 1.04E7 0.4875 0.39 0.00914 0 0.45 0.45 9.8462e-7 9.8462e-7
0.004 0.004 0.375 0.375
-14441.3 -5917.3 1816.0 1663.1 1273.5 673.3 2010.0 1
-14741.3 -5917.3 1913.5 1760.6 1370.7 770.8 2310.0
0.23 0.6 1 2

28 FRAME INTCOLUMN4
2 0 0 4 0 0 3
2.5E7 1.04E7 0.4875 0.39 0.00914 0 0.45 0.45 9.8462e-7 9.8462e-7
0.004 0.004 0.375 0.375
-14741.3 -5917.3 1913.5 1760.6 1370.7 770.8 2310.0 1
-14831.3 -5917.3 1942.7 1789.9 1400.0 800.4 2400.0
0.23 0.6 1 2

29 FRAME INTCOLUMN5
2 0 0 4 0 0 3
2.5E7 1.04E7 0.4875 0.39 0.00914 0 0.45 0.45 9.8462e-7 9.8462e-7
0.004 0.004 0.375 0.375
-14831.3 -5917.3 1942.7 1789.9 1400.0 800.4 2400.0 1
-14621.3 -5917.3 1874.5 1721.6 1331.7 731.8 2190.0
0.23 0.6 1 2

30 FRAME INTCOLUMN6
2 0 0 4 0 0 3
2.5E7 1.04E7 0.4875 0.39 0.00914 0 0.45 0.45 9.8462e-7 9.8462e-7
0.004 0.004 0.375 0.375
-14621.3 -5917.3 1874.5 1721.6 1331.7 731.8 2190.0 1
-14801.3 -5917.3 1933.0 1780.1 1390.4 790.5 2370.0
0.23 0.6 1 2

31 FRAME INTCOLUMN7
2 0 0 4 0 0 3
2.5E7 1.04E7 0.4875 0.39 0.00914 0 0.45 0.45 9.8462e-7 9.8462e-7
0.004 0.004 0.375 0.375
-14801.3 -5917.3 1933.0 1780.1 1390.4 790.5 2370.0 1
-14981.3 -5917.3 1991.5 1838.6 1448.6 849.2 2550.0
0.23 0.6 1 2

32 FRAME INTCOLUMN8
2 0 0 4 0 0 3
2.5E7 1.04E7 0.4875 0.39 0.00914 0 0.45 0.45 9.8462e-7 9.8462e-7
0.004 0.004 0.375 0.375
-14981.3 -5917.3 1991.5 1838.6 1448.6 849.2 2550.0 1
-15191.3 -5917.3 2059.7 1906.9 1517.0 916.9 2760.0
0.23 0.6 1 2

33 FRAME INTCOLUMN9
2 0 0 4 0 0 3
2.5E7 1.04E7 0.4875 0.39 0.00914 0 0.45 0.45 9.8462e-7 9.8462e-7

```
  0.004 0.004 0.375 0.375
  -15191.3 -5917.3 2059.7 1906.9 1517.0 916.9 2760.0 1
  -15401.3 -5917.3 2128.0 1975.1 1584.9 985.8 2970.0
  0.23 0.6 1 2

34 FRAME INTCOLUMN10
  2 0 0 4 0 0 3
  2.5E7 1.04E7 0.4875 0.39 0.00914 0 0.45 0.45 9.8462e-7 9.8462e-7
  0.004 0.004 0.375 0.375
  -15401.3 -5917.3 2128.0 1975.1 1584.9 985.8 2970.0 1
  -15731.3 -5917.3 2235.2 2082.4 1691.9 1092.5 3300.0
  0.23 0.6 1 2

35 FRAME INTCOLUMN11
  2 0 0 4 0 0 3
  2.5E7 1.04E7 0.4875 0.39 0.00914 0 0.45 0.45 9.8462e-7 9.8462e-7
  0.004 0.004 0.375 0.375
  -15731.3 -5917.3 2235.2 2082.4 1691.9 1092.5 3300.0 1
  -14891.3 -5917.3 1962.2 1809.4 1419.2 820.4 2460.0
  0.23 0.6 1 2

36 FRAME INTCOLUMN12
  2 0 0 4 0 0 3
  2.5E7 1.04E7 0.4875 0.39 0.00914 0 0.45 0.45 9.8462e-7 9.8462e-7
  0.004 0.004 0.375 0.375
  -14891.3 -5917.3 1962.2 1809.4 1419.2 820.4 2460.0 1
  -13601.3 -5917.3 1543.0 1390.1 1001.4 400.3 1170.0
  0.23 0.6 1 2

37 FRAME ADDITIONAL4PD
  2 3 0 0 0 0 0 0
  2.5E7 1.04E7 0.405 0.324 0.00615 0 0 0 0 0

WEIGHTS 0
1   0.0   0.0  0.0
2   0.0   0.0  0.0
3   0.0   0.0  0.0
4   0.0   0.0  0.0
5   550   0.0  0.0
6   550   0.0  0.0
7   550   0.0  0.0
8   550   0.0  0.0
9   550   0.0  0.0
10  550   0.0  0.0
11  550   0.0  0.0
12  550   0.0  0.0
13  550   0.0  0.0
14  550   0.0  0.0
15  550   0.0  0.0
16  550   0.0  0.0
17  550   0.0  0.0
18  550   0.0  0.0
19  550   0.0  0.0
20  550   0.0  0.0
21  550   0.0  0.0
22  550   0.0  0.0
23  550   0.0  0.0
24  550   0.0  0.0
25  550   0.0  0.0
26  550   0.0  0.0
27  550   0.0  0.0
28  550   0.0  0.0
29  550   0.0  0.0
30  550   0.0  0.0
31  550   0.0  0.0
32  550   0.0  0.0
33  550   0.0  0.0
34  550   0.0  0.0
35  550   0.0  0.0
36  550   0.0  0.0
37  550   0.0  0.0
38  550   0.0  0.0
39  550   0.0  0.0
40  550   0.0  0.0
41  550   0.0  0.0
42  550   0.0  0.0
43  550   0.0  0.0
44  550   0.0  0.0
45  550   0.0  0.0
46  550   0.0  0.0
47  550   0.0  0.0
48  550   0.0  0.0
49  550   0.0  0.0
50  550   0.0  0.0
51  550   0.0  0.0
52  550   0.0  0.0
65    0   0.0  0.0

LOADS
1        0          0    0
2        0          0    0
3        0          0    0
4        0          0    0
5        0     -76.55    0
6        0    -124.46    0
7        0    -124.46    0
8        0     -76.55    0
9        0     -76.55    0
10       0    -124.46    0
11       0    -124.46    0
12       0     -76.55    0
13       0     -76.55    0
```

| | | | |
|---|---|---|---|
| 14 | 0 | -124.46 | 0 |
| 15 | 0 | -124.46 | 0 |
| 16 | 0 | -76.55 | 0 |
| 17 | 0 | -76.55 | 0 |
| 18 | 0 | -124.46 | 0 |
| 19 | 0 | -124.46 | 0 |
| 20 | 0 | -76.55 | 0 |
| 21 | 0 | -76.55 | 0 |
| 22 | 0 | -124.46 | 0 |
| 23 | 0 | -124.46 | 0 |
| 24 | 0 | -76.55 | 0 |
| 25 | 0 | -76.55 | 0 |
| 26 | 0 | -124.46 | 0 |
| 27 | 0 | -124.46 | 0 |
| 28 | 0 | -76.55 | 0 |
| 29 | 0 | -76.55 | 0 |
| 30 | 0 | -124.46 | 0 |
| 31 | 0 | -124.46 | 0 |
| 32 | 0 | -76.55 | 0 |
| 33 | 0 | -76.55 | 0 |
| 34 | 0 | -124.46 | 0 |
| 35 | 0 | -124.46 | 0 |
| 36 | 0 | -76.55 | 0 |
| 37 | 0 | -76.55 | 0 |
| 38 | 0 | -124.46 | 0 |
| 39 | 0 | -124.46 | 0 |
| 40 | 0 | -76.55 | 0 |
| 41 | 0 | -76.55 | 0 |
| 42 | 0 | -124.46 | 0 |
| 43 | 0 | -124.46 | 0 |
| 44 | 0 | -76.55 | 0 |
| 45 | 0 | -76.55 | 0 |
| 46 | 0 | -124.46 | 0 |
| 47 | 0 | -124.46 | 0 |
| 48 | 0 | -76.55 | 0 |
| 49 | 0 | -62.23 | 0 |
| 50 | 0 | -110.14 | 0 |
| 51 | 0 | -110.14 | 0 |
| 52 | 0 | -62.23 | 0 |
| 53 | 0 | 0 | 0 |
| 54 | 0 | -1797.98 | 0 |
| 55 | 0 | -1797.98 | 0 |
| 56 | 0 | -1797.98 | 0 |
| 57 | 0 | -1797.98 | 0 |
| 58 | 0 | -1797.98 | 0 |
| 59 | 0 | -1797.98 | 0 |
| 60 | 0 | -1797.98 | 0 |
| 61 | 0 | -1797.98 | 0 |
| 62 | 0 | -1797.98 | 0 |
| 63 | 0 | -1797.98 | 0 |
| 64 | 0 | -1797.98 | 0 |
| 65 | 0 | -1855.26 | 0 |

EQUAKE L:\QUAKE\4203elc1.EQF
3 1 0.02 0.8333 27.10 0 0 1

## *F.4 Twelve Storey Structure B*

TWELVE STOREY B, THREE BAY, CONCRETE MOMENT RESISTING FRAME

* UNITS kN, M
* SELF WEIGHT IS INCLUDED IN 4400 KN PER FLOOR (2200 kN PER FRAME)
* FLEXIBILITIES ADDED AT END OF MEMBERS TO ACCOUNT FOR JOINT DEFORMATIONS
* UNIVERSITY OF CANTERBURY
* JOSE ANTONIO FLORES RUIZ

2 0 1 2 2 0 0 0 0 0 0
65 108 37 6 1 3 9.81 5.0 5.0 0.005 40 1
0 20 20 0 1 1 1 1 13 2 0
10 5 0.0001 0 0 0

NODES 0

| | | | | |
|---|---|---|---|---|
| 1 | 0 | 0.0 | 1 1 1 | 0 0 0 |
| 2 | 7 | 0.0 | 1 1 1 | 0 0 0 |
| 3 | 14 | 0.0 | 1 1 1 | 0 0 0 |
| 4 | 21 | 0.0 | 1 1 1 | 0 0 0 |
| 5 | 0 | 3.4 | 0 0 0 | 6 0 0 |
| 6 | 7 | 3.4 | 0 0 0 | 0 0 0 |
| 7 | 14 | 3.4 | 0 0 0 | 6 0 0 |
| 8 | 21 | 3.4 | 0 0 0 | 6 0 0 |
| 9 | 0 | 6.8 | 0 0 0 | 10 0 0 |
| 10 | 7 | 6.8 | 0 0 0 | 0 0 0 |
| 11 | 14 | 6.8 | 0 0 0 | 10 0 0 |
| 12 | 21 | 6.8 | 0 0 0 | 10 0 0 |
| 13 | 0 | 10.2 | 0 0 0 | 14 0 0 |
| 14 | 7 | 10.2 | 0 0 0 | 0 0 0 |
| 15 | 14 | 10.2 | 0 0 0 | 14 0 0 |
| 16 | 21 | 10.2 | 0 0 0 | 14 0 0 |
| 17 | 0 | 13.6 | 0 0 0 | 18 0 0 |
| 18 | 7 | 13.6 | 0 0 0 | 0 0 0 |
| 19 | 14 | 13.6 | 0 0 0 | 18 0 0 |
| 20 | 21 | 13.6 | 0 0 0 | 18 0 0 |
| 21 | 0 | 17.0 | 0 0 0 | 22 0 0 |
| 22 | 7 | 17.0 | 0 0 0 | 0 0 0 |
| 23 | 14 | 17.0 | 0 0 0 | 22 0 0 |
| 24 | 21 | 17.0 | 0 0 0 | 22 0 0 |
| 25 | 0 | 20.4 | 0 0 0 | 26 0 0 |
| 26 | 7 | 20.4 | 0 0 0 | 0 0 0 |
| 27 | 14 | 20.4 | 0 0 0 | 26 0 0 |
| 28 | 21 | 20.4 | 0 0 0 | 26 0 0 |
| 29 | 0 | 23.8 | 0 0 0 | 30 0 0 |

```
30   7  23.8  0 0 0    0 0 0
31  14  23.8  0 0 0   30 0 0
32  21  23.8  0 0 0   30 0 0
33   0  27.2  0 0 0   34 0 0
34   7  27.2  0 0 0    0 0 0
35  14  27.2  0 0 0   34 0 0
36  21  27.2  0 0 0   34 0 0
37   0  30.6  0 0 0   38 0 0
38   7  30.6  0 0 0    0 0 0
39  14  30.6  0 0 0   38 0 0
40  21  30.6  0 0 0   38 0 0
41   0  34.0  0 0 0   42 0 0
42   7  34.0  0 0 0    0 0 0
43  14  34.0  0 0 0   42 0 0
44  21  34.0  0 0 0   42 0 0
45   0  37.4  0 0 0   46 0 0
46   7  37.4  0 0 0    0 0 0
47  14  37.4  0 0 0   46 0 0
48  21  37.4  0 0 0   46 0 0
49   0  40.8  0 0 0   50 0 0
50   7  40.8  0 0 0    0 0 0
51  14  40.8  0 0 0   50 0 0
52  21  40.8  0 0 0   50 0 0
53  28   0.0  1 1 1    0 0 0
54  28   3.4  0 0 1    0 0 0
55  28   6.8  0 0 1    0 0 0
56  28  10.2  0 0 1    0 0 0
57  28  13.6  0 0 1    0 0 0
58  28  17.0  0 0 1    0 0 0
59  28  20.4  0 0 1    0 0 0
60  28  23.8  0 0 1    0 0 0
61  28  27.2  0 0 1    0 0 0
62  28  30.6  0 0 1    0 0 0
63  28  34.0  0 0 1    0 0 0
64  28  37.4  0 0 1    0 0 0
65  28  40.8  0 0 1    0 0 0

DRIFT

4 8 12 16 20 24 28 32 36 40 44 48 52

ELEMENTS 0
1   1    5    6  0 0 0        !BEAM LEVEL 1
2   1    6    7  0 0 0
3   1    7    8  0 0 0
4   2    9   10  0 0 0        !BEAM LEVEL 2
5   2   10   11  0 0 0
6   2   11   12  0 0 0
7   3   13   14  0 0 0        !BEAM LEVEL 3
8   3   14   15  0 0 0
9   3   15   16  0 0 0
10   4   17   18  0 0 0       !BEAM LEVEL 4
11   4   18   19  0 0 0
12   4   19   20  0 0 0
13   5   21   22  0 0 0       !BEAM LEVEL 5
14   5   22   23  0 0 0
15   5   23   24  0 0 0
16   6   25   26  0 0 0       !BEAM LEVEL 6
17   6   26   27  0 0 0
18   6   27   28  0 0 0
19   7   29   30  0 0 0       !BEAM LEVEL 7
20   7   30   31  0 0 0
21   7   31   32  0 0 0
22   8   33   34  0 0 0       !BEAM LEVEL 8
23   8   34   35  0 0 0
24   8   35   36  0 0 0
25   9   37   38  0 0 0       !BEAM LEVEL 9
26   9   38   39  0 0 0
27   9   39   40  0 0 0
28  10   41   42  0 0 0       !BEAM LEVEL 10
29  10   42   43  0 0 0
30  10   43   44  0 0 0
31  11   45   46  0 0 0       !BEAM LEVEL 11
32  11   46   47  0 0 0
33  11   47   48  0 0 0
34  12   49   50  0 0 0       !BEAM LEVEL 12
35  12   50   51  0 0 0
36  12   51   52  0 0 0
37  13    1    5  0 0 0       !EXT COL GROUND LEVEL
38  14    5    9  0 0 0
39  15    9   13  0 0 0
40  16   13   17  0 0 0
41  17   17   21  0 0 0
42  18   21   25  0 0 0
43  19   25   29  0 0 0
44  20   29   33  0 0 0
45  21   33   37  0 0 0
46  22   37   41  0 0 0
47  23   41   45  0 0 0
48  24   45   49  0 0 0
49  25    2    6  0 0 0       !INT COL GROUND LEVEL
50  26    6   10  0 0 0
51  27   10   14  0 0 0
52  28   14   18  0 0 0
53  29   18   22  0 0 0
54  30   22   26  0 0 0
55  31   26   30  0 0 0
56  32   30   34  0 0 0
57  33   34   38  0 0 0
58  34   38   42  0 0 0
59  35   42   46  0 0 0
60  36   46   50  0 0 0
```

```
61  25   3   7  0 0 0        !INT COL GROUND LEVEL
62  26   7  11  0 0 0
63  27  11  15  0 0 0
64  28  15  19  0 0 0
65  29  19  23  0 0 0
66  30  23  27  0 0 0
67  31  27  31  0 0 0
68  32  31  35  0 0 0
69  33  35  39  0 0 0
70  34  39  43  0 0 0
71  35  43  47  0 0 0
72  36  47  51  0 0 0
73  13   4   8  0 0 0        !EXT COL GROUND LEVEL
74  14   8  12  0 0 0
75  15  12  16  0 0 0
76  16  16  20  0 0 0
77  17  20  24  0 0 0
78  18  24  28  0 0 0
79  19  28  32  0 0 0
80  20  32  36  0 0 0
81  21  36  40  0 0 0
82  22  40  44  0 0 0
83  23  44  48  0 0 0
84  24  48  52  0 0 0
85  37   8  54  0 0 0        !ADDITIONAL ELEMENTS FOR P-DELTA
86  37  12  55  0 0 0
87  37  16  56  0 0 0
88  37  20  57  0 0 0
89  37  24  58  0 0 0
90  37  28  59  0 0 0
91  37  32  60  0 0 0
92  37  36  61  0 0 0
93  37  40  62  0 0 0
94  37  44  63  0 0 0
95  37  48  64  0 0 0
96  37  52  65  0 0 0
97  37  53  54  0 0 0
98  37  54  55  0 0 0
99  37  55  56  0 0 0
100 37  56  57  0 0 0
101 37  57  58  0 0 0
102 37  58  59  0 0 0
103 37  59  60  0 0 0
104 37  60  61  0 0 0
105 37  61  62  0 0 0
106 37  62  63  0 0 0
107 37  63  64  0 0 0
108 37  64  65  0 0 0

PROPS

1 FRAME LEVEL1
 1 0 0 4 0 0 0 0
 2.5E7 1.04E7 0.5825 0.466 0.0158 13.69 0.3875 0.3875 4.9041e-7 4.9041e-7
 0.004 0.004 0.45 0.45
 1580.0 -12624.1 632.0 -632.0 632.0 -632.0
 0.35 0.43 1 2

2 FRAME LEVEL2
 1 0 0 4 0 0 0 0
 2.5E7 1.04E7 0.5825 0.466 0.0158 13.69 0.3875 0.3875 4.9041e-7 4.9041e-7
 0.004 0.004 0.45 0.45
 1669.0 -12624.2 667.6 -667.6 667.6 -667.6
 0.35 0.43 1 2

3 FRAME LEVEL3
 1 0 0 4 0 0 0 0
 2.5E7 1.04E7 0.5825 0.466 0.0158 13.69 0.3875 0.3875 4.9041e-7 4.9041e-7
 0.004 0.004 0.45 0.45
 1548.5 -12624.0 619.4 -619.4 619.4 -619.4
 0.35 0.43 1 2

4 FRAME LEVEL4
 1 0 0 4 0 0 0 0
 2.5E7 1.04E7 0.5825 0.466 0.0158 13.69 0.3875 0.3875 4.9041e-7 4.9041e-7
 0.004 0.004 0.45 0.45
 1403.8 -12623.9 561.5 -561.5 561.5 -561.5
 0.35 0.43 1 2

5 FRAME LEVEL5
 1 0 0 4 0 0 0 0
 2.5E7 1.04E7 0.5825 0.466 0.0158 13.69 0.3875 0.3875 4.9041e-7 4.9041e-7
 0.004 0.004 0.45 0.45
 1273.5 -12623.8 509.4 -509.4 509.4 -509.4
 0.35 0.43 1 2

6 FRAME LEVEL6
 1 0 0 4 0 0 0 0
 2.5E7 1.04E7 0.5825 0.466 0.0158 13.69 0.3875 0.3875 4.9041e-7 4.9041e-7
 0.004 0.004 0.45 0.45
 1273.5 -12623.8 509.4 -509.4 509.4 -509.4
 0.35 0.43 1 2

7 FRAME LEVEL7
 1 0 0 4 0 0 0 0
  2.5E7 1.04E7 0.500 0.400 0.00929 11.75 0.350 0.350 7.5318e-7 7.5319e-7
 0.004 0.004 0.375 0.375
 1252.0 -10520.0 406.9 -406.9 406.9 -406.9
 0.35 0.43 1 2

8 FRAME LEVEL8
```

```
1 0 0 4 0 0 0 0
2.5E7 1.04E7 0.500 0.400 0.00929 11.75 0.350 0.350 7.5318e-7 7.5319e-7
0.004 0.004 0.375 0.375
1138.2 -10519.9 369.9 -369.9 369.9 -369.9
0.35 0.43 1 2

9 FRAME LEVEL9
1 0 0 4 0 0 0 0
2.5E7 1.04E7 0.500 0.400 0.00929 11.75 0.350 0.350 7.5318e-7 7.5319e-7
0.004 0.004 0.375 0.375
1138.2 -10519.9 369.9 -369.9 369.9 -369.9
0.35 0.43 1 2

10 FRAME LEVEL10
1 0 0 4 0 0 0 0
2.5E7 1.04E7 0.500 0.400 0.00929 11.75 0.350 0.350 7.5318e-7 7.5319e-7
0.004 0.004 0.375 0.375
1138.2 -10519.9 369.9 -369.9 369.9 -369.9
0.35 0.43 1 2

11 FRAME LEVEL11
1 0 0 4 0 0 0 0
2.5E7 1.04E7 0.500 0.400 0.00929 11.75 0.350 0.350 7.5318e-7 7.5319e-7
0.004 0.004 0.375 0.375
1138.2 -10519.9 369.9 -369.9 369.9 -369.9
0.35 0.43 1 2

12 FRAME LEVEL12
1 0 0 4 0 0 0 0
2.5E7 1.04E7 0.500 0.400 0.00929 11.75 0.350 0.350 7.5318e-7 7.5319e-7
0.004 0.004 0.375 0.375
1138.2 -10519.9 369.9 -369.9 369.9 -369.9
0.35 0.43 1 2

13 FRAME EXTCOLUMN1
2 0 0 4 0 0 3 0
2.5E7 1.04E7 0.50375 0.403 0.0101 11.84 0 0.45 0 8.9237e-7
0.004 0.004 0.3875 0.3875
-15575.6 -6128.6 2163.2 2001.0 1585.6 942.0 2730.0 1
-14975.6 -6128.6 1960.7 1798.5 1383.4 739.2 2130.0
0.23 0.6 1 2

14 FRAME EXTCOLUMN2
2 0 0 4 0 0 3 0
2.5E7 1.04E7 0.50375 0.403 0.0101 11.84 0.45 0.45 8.9237e-7 8.9237e-7
0.004 0.004 0.3875 0.3875
-14975.6 -6128.6 1960.7 1798.5 1383.4 739.2 2130.0 1
-15215.6 -6128.6 2041.7 1879.5 1464.3 820.1 2370.0
0.23 0.6 1 2

15 FRAME EXTCOLUMN3
2 0 0 4 0 0 3 0
2.5E7 1.04E7 0.50375 0.403 0.0101 11.84 0.45 0.45 8.9237e-7 8.9237e-7
0.004 0.004 0.3875 0.3875
-15215.6 -6128.6 2041.7 1879.5 1464.3 820.1 2370.0 1
-15275.6 -6128.6 2061.9 1899.8 1484.5 840.8 2430.0
0.23 0.6 1 2

16 FRAME EXTCOLUMN4
2 0 0 4 0 0 3 0
2.5E7 1.04E7 0.50375 0.403 0.0101 11.84 0.45 0.45 8.9237e-7 8.9237e-7
0.004 0.004 0.3875 0.3875
-15275.6 -6128.6 2061.9 1899.8 1484.5 840.8 2430.0 1
-15155.6 -6128.6 2021.4 1859.3 1444.2 799.7 2310.0
0.23 0.6 1 2

17 FRAME EXTCOLUMN5
2 0 0 4 0 0 3 0
2.5E7 1.04E7 0.50375 0.403 0.0101 11.84 0.45 0.45 8.9237e-7 8.9237e-7
0.004 0.004 0.3875 0.3875
-15155.6 -6128.6 2021.4 1859.3 1444.2 799.7 2310.0 1
-14945.6 -6128.6 1950.6 1788.4 1373.3 729.2 2100.0
0.23 0.6 1 2

18 FRAME EXTCOLUMN6
2 0 0 4 0 0 3 0
2.5E7 1.04E7 0.50375 0.403 0.0101 11.84 0.45 0.45 8.9237e-7 8.9237e-7
0.004 0.004 0.3875 0.3875
-14945.6 -6128.6 1950.6 1788.4 1373.3 729.2 2100.0 1
-15035.6 -6128.6 1980.9 1818.8 1403.5 759.2 2190.0
0.23 0.6 1 2

19 FRAME EXTCOLUMN7
2 0 0 4 0 0 3 0
2.5E7 1.04E7 0.455 0.364 0.00743 10.69 0.375 0.375 1.0092e-6 1.0092e-6
0.004 0.004 0.35 0.35
-13792.5 -5494.6 1669.4 1534.3 1193.0 677.1 2190.0 1
-13312.5 -5494.3 1525.4 1390.3 1049.6 533.8 1710.0
0.23 0.6 1 2

20 FRAME EXTCOLUMN8
2 0 0 4 0 0 3 0
2.5E7 1.04E7 0.455 0.364 0.00743 10.69 0.375 0.375 1.0092e-6 1.0092e-6
0.004 0.004 0.35 0.35
-13312.5 -5494.3 1525.4 1390.3 1049.6 533.8 1710.0 1
-13312.5 -5494.6 1525.4 1390.3 1049.6 533.8 1710.0
0.23 0.6 1 2

21 FRAME EXTCOLUMN9
2 0 0 4 0 0 3 0
2.5E7 1.04E7 0.455 0.364 0.00743 10.69 0.375 0.375 1.0092e-6 1.0092e-6
0.004 0.004 0.35 0.35
```

-13312.5 -5494.3 1525.4 1390.3 1049.6 533.8 1710.0 1
-13372.5 -5494.6 1543.4 1408.3 1067.5 551.5 1770.0
0.23 0.6 1 2

22 FRAME EXTCOLUMN10
2 0 0 4 0 0 3 0
2.5E7 1.04E7 0.455 0.364 0.00743 10.69 0.375 0.375 1.0092e-6 1.0092e-6
0.004 0.004 0.35 0.35
-13372.5 -5494.6 1543.4 1408.3 1067.5 551.5 1770.0 1
-13432.5 -5494.6 1561.4 1426.3 1085.4 569.4 1830.0
0.23 0.6 1 2

23 FRAME EXTCOLUMN11
2 0 0 4 0 0 3 0
2.5E7 1.04E7 0.455 0.364 0.00743 10.69 0.375 0.375 1.0092e-6 1.0092e-6
0.004 0.004 0.35 0.35
-13432.5 -5494.6 1561.4 1426.3 1085.4 569.4 1830.0 1
-13132.5 -5494.6 1471.4 1336.3 996.0 478.7 1530.0
0.23 0.6 1 2

24 FRAME EXTCOLUMN12
2 0 0 4 0 0 3 0
2.5E7 1.04E7 0.455 0.364 0.00743 10.69 0.375 0.375 1.0092e-6 1.0092e-6
0.004 0.004 0.35 0.35
-13132.5 -5494.6 1471.4 1336.3 996.0 478.7 1530.0 1
-11872.5 -5494.6 1093.4 958.3 620.2 92.0 270.0
0.23 0.6 1 2

25 FRAME INTCOLUMNS1
2 0 0 4 0 0 3 0
2.5E7 1.04E7 0.50375 0.403 0.0101 11.84 0 0.45 0 8.9237e-7
0.004 0.004 0.3875 0.3875
-13625.6 -6128.6 1505.1 1342.9 928.4 280.9 780.0 1
-14345.6 -6128.6 1748.1 1585.9 1171.3 525.8 1500.0
0.23 0.6 1 2

26 FRAME INTCOLUMNS2
2 0 0 4 0 0 3 0
2.5E7 1.04E7 0.50375 0.403 0.0101 11.84 0.45 0.45 8.9237e-7 8.9237e-7
0.004 0.004 0.3875 0.3875
-14345.6 -6128.6 1748.1 1585.9 1171.3 525.8 1500.0 1
-14675.6 -6128.6 1859.4 1697.3 1282.4 638.0 1830.0
0.23 0.6 1 2

27 FRAME INTCOLUMNS3
2 0 0 4 0 0 3 0
2.5E7 1.04E7 0.50375 0.403 0.0101 11.84 0.45 0.45 8.9237e-7 8.9237e-7
0.004 0.004 0.3875 0.3875
-14675.6 -6128.6 1859.4 1697.3 1282.4 638.0 1830.0 1
-14975.6 -6128.6 1960.7 1798.5 1383.4 739.2 2130.0
0.23 0.6 1 2

28 FRAME INTCOLUMNS4
2 0 0 4 0 0 3 0
2.5E7 1.04E7 0.50375 0.403 0.0101 11.84 0.45 0.45 8.9237e-7 8.9237e-7
0.004 0.004 0.3875 0.3875
-14975.6 -6128.6 1960.7 1798.5 1383.4 739.2 2130.0 1
-15035.6 -6128.6 1980.9 1818.8 1403.5 759.2 2190.0
0.23 0.6 1 2

29 FRAME INTCOLUMNS5
2 0 0 4 0 0 3 0
2.5E7 1.04E7 0.50375 0.403 0.0101 11.84 0.45 0.45 8.9237e-7 8.9237e-7
0.004 0.004 0.3875 0.3875
-15035.6 -6128.6 1980.9 1818.8 1403.5 759.2 2190.0 1
-14855.6 -6128.6 1920.2 1758.0 1343.2 698.4 2010.0
0.23 0.6 1 2

30 FRAME INTCOLUMNS6
2 0 0 4 0 0 3 0
2.5E7 1.04E7 0.50375 0.403 0.0101 11.84 0.45 0.45 8.9237e-7 8.9237e-7
0.004 0.004 0.3875 0.3875
-14855.6 -6128.6 1920.2 1758.0 1343.2 698.4 2010.0 1
-15365.6 -6128.6 2092.3 1930.2 1515.2 870.6 2520.0
0.23 0.6 1 2

31 FRAME INTCOLUMNS7
2 0 0 4 0 0 3 0
2.5E7 1.04E7 0.455 0.364 0.00743 10.69 0.375 0.375 1.0092e-6 1.0092e-6
0.004 0.004 0.35 0.35
-14122.5 -5494.6 1768.4 1633.3 1291.6 776.2 2520.0 1
-13552.5 -5494.6 1597.4 1462.3 1121.2 605.8 1950.0
0.23 0.6 1 2

32 FRAME INTCOLUMNS8
2 0 0 4 0 0 3 0
2.5E7 1.04E7 0.455 0.364 0.00743 10.69 0.375 0.375 1.0092e-6 1.0092e-6
0.004 0.004 0.35 0.35
-13552.5 -5494.6 1597.4 1462.3 1121.2 605.8 1950.0 1
-13642.5 -5494.6 1624.4 1489.3 1147.9 633.0 2040.0
0.23 0.6 1 2

33 FRAME INTCOLUMNS9
2 0 0 4 0 0 3 0
2.5E7 1.04E7 0.455 0.364 0.00743 10.69 0.375 0.375 1.0092e-6 1.0092e-6
0.004 0.004 0.35 0.35
-13642.5 -5494.6 1624.4 1489.3 1147.9 633.0 2040.0 1
-13882.5 -5494.6 1696.4 1561.3 1219.7 704.1 2280.0
0.23 0.6 1 2

34 FRAME INTCOLUMNS10
2 0 0 4 0 0 3 0
2.5E7 1.04E7 0.455 0.364 0.00743 10.69 0.375 0.375 1.0092e-6 1.0092e-6
0.004 0.004 0.35 0.35
-13882.5 -5494.6 1696.4 1561.3 1219.7 704.1 2280.0 1
-14182.5 -5494.6 1786.4 1651.3 1309.6 794.8 2580.0
0.23 0.6 1 2

35 FRAME INTCOLUMNS11
2 0 0 4 0 0 3 0
2.5E7 1.04E7 0.455 0.364 0.00743 10.69 0.375 0.375 1.0092e-6 1.0092e-6
0.004 0.004 0.35 0.35
-14182.5 -5494.6 1786.4 1651.3 1309.6 794.8 2580.0 1
-13582.5 -5494.6 1606.4 1471.3 1130.0 614.4 1980.0
0.23 0.6 1 2

36 FRAME INTCOLUMNS12
2 0 0 4 0 0 3 0
2.5E7 1.04E7 0.455 0.364 0.00743 10.69 0.375 0.375 1.0092e-6 1.0092e-6
0.004 0.004 0.35 0.35
-13582.5 -5494.6 1606.4 1471.3 1130.0 614.4 1980.0 1
-11902.5 +5494.6 1102.4 967.3 629.2 102.3 300.0
0.23 0.6 1 2

37 FRAME ADITIONAL4PD
2 3 0 0 0 0 0 0
2.5E7 1.04E7 0.405 0.324 0.00615 0 0 0 0 0

WEIGHTS 0

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| 2 | 0 | 0 | 0 |
| 3 | 0 | 0 | 0 |
| 4 | 0 | 0 | 0 |
| 5 | 448.53 | 0 | 0 |
| 6 | 448.53 | 0 | 0 |
| 7 | 448.53 | 0 | 0 |
| 8 | 448.53 | 0 | 0 |
| 9 | 448.53 | 0 | 0 |
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| 11 | 448.53 | 0 | 0 |
| 12 | 448.53 | 0 | 0 |
| 13 | 448.53 | 0 | 0 |
| 14 | 448.53 | 0 | 0 |
| 15 | 448.53 | 0 | 0 |
| 16 | 448.53 | 0 | 0 |
| 17 | 448.53 | 0 | 0 |
| 18 | 448.53 | 0 | 0 |
| 19 | 448.53 | 0 | 0 |
| 20 | 448.53 | 0 | 0 |
| 21 | 448.53 | 0 | 0 |
| 22 | 448.53 | 0 | 0 |
| 23 | 448.53 | 0 | 0 |
| 24 | 448.53 | 0 | 0 |
| 25 | 448.53 | 0 | 0 |
| 26 | 448.53 | 0 | 0 |
| 27 | 448.53 | 0 | 0 |
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| 44 | 459.97 | 0 | 0 |
| 45 | 459.97 | 0 | 0 |
| 46 | 459.97 | 0 | 0 |
| 47 | 459.97 | 0 | 0 |
| 48 | 459.97 | 0 | 0 |
| 49 | 474.14 | 0 | 0 |
| 50 | 474.14 | 0 | 0 |
| 51 | 474.14 | 0 | 0 |
| 52 | 474.14 | 0 | 0 |
| 53 | 0 | 0 | 0 |
| 54 | 0 | 0 | 0 |
| 55 | 0 | 0 | 0 |
| 56 | 0 | 0 | 0 |
| 57 | 0 | 0 | 0 |
| 58 | 0 | 0 | 0 |
| 59 | 0 | 0 | 0 |
| 60 | 0 | 0 | 0 |
| 61 | 0 | 0 | 0 |
| 62 | 0 | 0 | 0 |
| 63 | 0 | 0 | 0 |
| 64 | 0 | 0 | 0 |
| 65 | 0 | 0 | 0 |

LOADS

| | | | |
|---|---|---|---|
| 1 | 0 | 0 | 0 |
| 2 | 0 | 0 | 0 |
| 3 | 0 | 0 | 0 |
| 4 | 0 | 0 | 0 |
| 5 | 0 | -77.51 | 0 |
| 6 | 0 | -125.42 | 0 |

| | | | |
|---|---|---|---|
| 7 | 0 | -125.42 | 0 |
| 8 | 0 | -77.51 | 0 |
| 9 | 0 | -77.51 | 0 |
| 10 | 0 | -125.42 | 0 |
| 11 | 0 | -125.42 | 0 |
| 12 | 0 | -77.51 | 0 |
| 13 | 0 | -77.51 | 0 |
| 14 | 0 | -125.42 | 0 |
| 15 | 0 | -125.42 | 0 |
| 16 | 0 | -77.51 | 0 |
| 17 | 0 | -77.51 | 0 |
| 18 | 0 | -125.42 | 0 |
| 19 | 0 | -125.42 | 0 |
| 20 | 0 | -77.51 | 0 |
| 21 | 0 | -77.51 | 0 |
| 22 | 0 | -125.42 | 0 |
| 23 | 0 | -125.42 | 0 |
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| 27 | 0 | -125.42 | 0 |
| 28 | 0 | -77.51 | 0 |
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| 30 | 0 | -110.59 | 0 |
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| 32 | 0 | -69.46 | 0 |
| 33 | 0 | -69.46 | 0 |
| 34 | 0 | -110.59 | 0 |
| 35 | 0 | -110.59 | 0 |
| 36 | 0 | -69.46 | 0 |
| 37 | 0 | -69.46 | 0 |
| 38 | 0 | -110.59 | 0 |
| 39 | 0 | -110.59 | 0 |
| 40 | 0 | -69.46 | 0 |
| 41 | 0 | -69.46 | 0 |
| 42 | 0 | -110.59 | 0 |
| 43 | 0 | -110.59 | 0 |
| 44 | 0 | -69.46 | 0 |
| 45 | 0 | -69.46 | 0 |
| 46 | 0 | -110.59 | 0 |
| 47 | 0 | -110.59 | 0 |
| 48 | 0 | -69.46 | 0 |
| 49 | 0 | -55.29 | 0 |
| 50 | 0 | -96.42 | 0 |
| 51 | 0 | -96.42 | 0 |
| 52 | 0 | -55.29 | 0 |
| 53 | 0 | 0 | 0 |
| 54 | 0 | -1794.14 | 0 |
| 55 | 0 | -1794.14 | 0 |
| 56 | 0 | -1794.14 | 0 |
| 57 | 0 | -1794.14 | 0 |

| | | | |
|---|---|---|---|
| 58 | 0 | -1794.14 | 0 |
| 59 | 0 | -1794.14 | 0 |
| 60 | 0 | -1839.90 | 0 |
| 61 | 0 | -1839.90 | 0 |
| 62 | 0 | -1839.90 | 0 |
| 63 | 0 | -1839.90 | 0 |
| 64 | 0 | -1839.90 | 0 |
| 65 | 0 | -1896.58 | 0 |

EQUAKE L:\QUAKE\4203elc1.EQF
3 1 0.02 0.8333 27.10 0 0 1